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OHIO STATE UNIVERSITY
CIVIL ENGINEERING PUBLICATIONS
NUMBER THREE

DESIGNING AND DETAILING OF SIMPLE STEEL STRUCTURES

BY

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FIRST EDITION

COLUMBUS, OHIO
1909

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THE HANN & ADAIR PRINTING CO., COLUMBUS, OHIO

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PREFACE

The object sought in this book is to collect from the many larger and more exhaustive works on structural steel design, those parts which are applicable to simple structures, and which can be taken up in technical schools in the limited time usually allotted to the subject; and at the same time, to show by general cases and specific examples how the simple laws of statics may be applied to the details of steel structures with the object of producing details which are in accord with the stresses they have to transmit.

It is presumed that the student has already finished a course in stresses, and little time is given here to the methods of calculating the primary stresses in structures.

An effort has been made to make the nomenclature, throughout, conform to that used in *"Stresses in Structures,"* by Prof. A. H. Heller, and a table is given so that the meaning of any letter or character in any formula can be at once determined by reference to it. In some cases where reference is made to another book, and a formula is taken bodily from it, the nomenclature of the original author is retained and the meaning of the letters given in connection.

Cross references to other articles in this book are indicated by figures in parentheses giving the article number, thus (14). References to other works on the subject are given in foot notes.

The author wishes to acknowledge his indebtedness to Mr. C. C. Heller for the privilege of using various manuscript notes and sketches, left at his death by Prof. A. H. Heller, which have formed the basis of many of the articles in this book.

It is hoped that by the illustrations given and the methods employed, the reasons will be made apparent for many of the details commonly employed in structural work, and which are many times put in by "rule of thumb" and too often without due consideration of the stresses they have to carry.

CLYDE T. MORRIS.

Columbus, O., April 6, 1909.

TABLE OF CONTENTS

PAGE

Chapter I—Riveting 1
Art. 1 Dimensions of Rivets 1
Art. 2 Rivet Holes 2
Art. 3 Driving Rivets 3
Art. 4 Theory of Riveting 4
Art. 5 Requirements for a good Riveted Joint 9
Art. 6 Proper Sizes of Rivets 9
Art. 7 Spacing of Rivets 10
Art. 8 Kinds of Joints 12
Art. 9 Design of Riveted Connections 13
Art. 10 Examples of Riveted Joints 15
Art. 11 Net-sections of Tension Members 20
Art. 12 Eccentric Stresses in Riveted Connections 25
Art. 13 Showing Rivets on Drawings 28

Chapter II—Designing and Estimating 29
Art. 14 Kinds of Structural Steel Work 29
Art. 15 Kinds of Shops 30
Art. 16 Proposals and Contracts 30
Art. 17 Designs and Estimates 31
Art. 18 Time Savers 35
Art. 19 Order of Estimating 37
Art. 20 Specifications 41
Art. 21 Stress Sheets and General Plans 42

Chapter III—Manufacture and Erection 44
Art. 22 Shop Operations 44
Art. 23 Erection 45
Art. 24 Drafting Department 46
Art. 25 A Draftsman's Equipment 47
Art. 26 Ordering Materials 51
Art. 27 Shop Drawings 54
Art. 28 Order of Proceedure for a Pin-Connected Bridge 62
Art. 29 Order of Proceedure for a Plate Girder Bridge 65

PAGE

Art. 30 Shop Bills 67
Art. 31 Shipment 69
Art. 32 Materials 70
Art. 33 Inspection 75

Chapter IV—Roofs 77
Art. 34 Construction 77
Art. 35 Roof Coverings 77
Art. 36 Types of Trusses 78
Art. 37 Building Construction 80
Art. 38 Loads 81
Art. 39 Stresses 84
Art. 40 The Design of a Roof 84
Art. 41 The Detail Drawings 91

Chapter V—Plate Girder Bridges 99
Art. 42 Construction and Uses 99
Art. 43 Stresses in Girders 99
Art. 44 The Web 100
Art. 45 The Flanges 102
Art. 46 Economic Depth 103
Art. 47 Stiffeners 105
Art. 48 Web Splices 106
Art. 49 Flange Riveting 107
Art. 50 Flange Splices 110
Art. 51 Design of a Stringer 110
Art. 52 Design of a Deck Plate Girder Bridge 115
Art. 53 Through Plate Girders 137

Chapter VI—Pin-Connected Bridges 139
Art. 54 Construction 139
Art. 55 Types of Trusses 139
Art. 56 Loads 140
Art. 57 Tension Members 141
Art. 58 Compression Members 142
Art. 59 Lateral Systems 146
Art. 60 Design of a Pin-Connected Railway Bridge 146
Art. 61 Dead Load 147
Art. 62 The Depth 147

PAGE
Art. 63 Stresses 148
Art. 64 Design of Tension Members 150
Art. 65 Design of Compression Members 154
Art. 66 Design of the End Posts 159
Art. 67 The Portal Bracing 163
Art. 68 Design of Floor Beams 165
Art. 69 Top Lateral Bracing 169
Art. 70 Bottom Lateral Bracing 170
Art. 71 Shoes and Rollers 171
Art. 72 Estimate and Stress Sheet................173

Chapter VII—Details of Pin-Connected Bridges...........177
Art. 73 Pins 177
Art. 74 Calculation of Pins 178
Art. 75 Details of a Riveted Tension Member........182
Art. 76 Location of Pins in Top Chord and End Post..185
Art. 77 Lacing of Compression Members............187
Art. 78 Details of the Floor Beams................190

NOTATION

A=total area of cross-section (square inches).
A_F=net area of one flange.
A_w=gross area of cross section of web=th.
a=distance shown in the figure.
b=distance shown in the figure.
C=Centrifugal force per pound.
C C_1 C_2 C' etc.=Constants of integration.
c=distance shown in figure.
D=direct stress.
$D.L.$=dead load or dead load stress.
d=distance from neutral axis to a parallel axis.
=depth between centers of gravity of the flanges of a girder.
=depth between centers of chords of a truss.
E=Modulus of elasticity.
e=distance shown in the figure.
=eccentricity of application of load.
H=horizontal reaction.
h=depth of the web of a girder, (inches).
I=Moment of Inertia.
k=distance shown in the figure.
k_1=distance between centers of bearings at the top of post.
k_2=distance between centers of bearings at the bottom of post.
L=total length.
$L.L.$=live load or live load stress.
l_1 l_2 etc.=partial lengths.
M=moment about any point or bending moment.
N=number of panels.
P=concentrated load or force.
p=panel length.
R=reaction.
=resultant of two or more forces.
r=radius of gyration.
S=shear.
s=unit stress.
s_1=maximum unit stress in extreme fiber.

s_c=unit stress in compression$=\dfrac{P}{A}$

s_D=dead load unit stress.

s_L=live load unit stress.

s_t=unit stress in tension.

s_w=working unit stress.

t=thikness of web of plate girder.

V=vertical reaction due to horizontal forces.

v=distance perpendicular to the neutral axis.

W=total uniform load.

w=load per foot.

θ=angle shown in figure.

=Angle with the vertical made by a diagonal truss member.

CHAPTER I.

RIVETING.

Rivets are used not only to connect members of riveted structures together, but also in all sorts of steel structures, to join parts of members built up of plates, angles, channels, etc., and for connecting details, such as pin plates, lacing bars and batten plates.

1. Dimensions of Rivets. Rivets are made in a machine, which upsets one end of a hot bar of steel or iron, forming the head, and cuts off enough of the bar to make a rivet of the desired length.

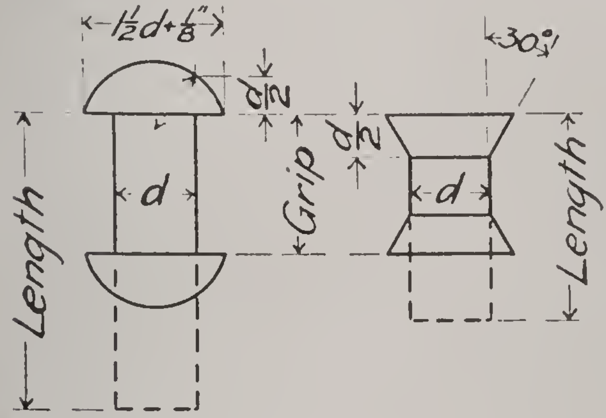
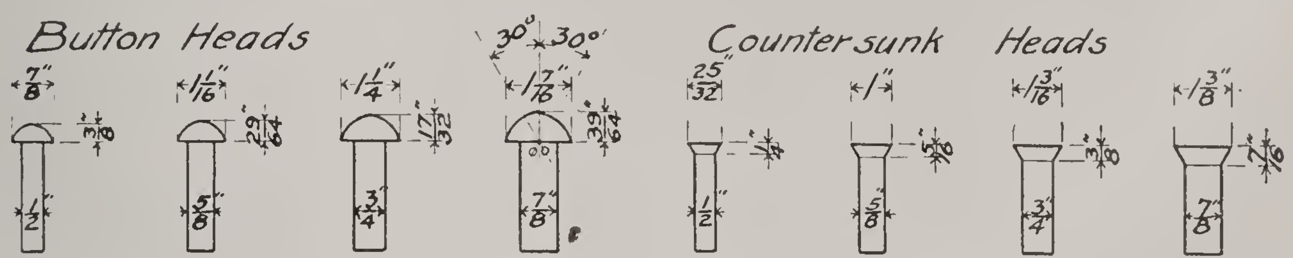


Fig. 1.

The size of a rivet is designated by the diameter of the shank, and its length under head, thus ¾ in. x 3¼ in.

"Button heads," which are hemispherical are used where there is room for them. Their size depends upon the diameter of the rivet. Fig. 1 shows about the proportions which are used in structural work for "button heads" and countersunk rivets.

Fig. 2 gives the standard heads used by The American Bridge Company. It will be noted that the heads do not have spherical surfaces before driving. The cups on the riveting machine are supposed to have hollow spherical surfaces, so that



American Bridge Company's Standard Rivets.
Fig. 2.

the pressure will at first be concentrated on the center of the head when the rivet is being driven. This aids in upsetting the body of the rivet so that it will fill the hole.

The grip of a rivet is equal to the sum of the thicknesses of the pieces joined. The distance between the heads of a rivet

will usually exceed the grip, on account of the roughness of the surfaces of the parts joined. Allowance is made for this in the length of the rivet used, which must be long enough so that there will be sufficient metal to fill the hole and form the head. A table of lengths required for different grips is given in the hand-books published by the various steel companies. (See *Cambria*, page 337).[1]

2. Rivet Holes. Holes for rivets are either *punched*, *sub-punched and reamed* or *drilled*. When reaming is required, the amount varies with different specifications from 1/8 in. to 1/4 in. That is, the diameter of the hole punched is from 1/8 in. to 1/4 in. smaller than the finished hole. The object of reaming is to remove the material surrounding the hole which is more or less injured in punching, and to insure a better *fit* and *matching of holes*. The injury done in punching is greater in thick material than in thin, and in medium steel than in soft steel. Hard steel is seldom used in structural work.

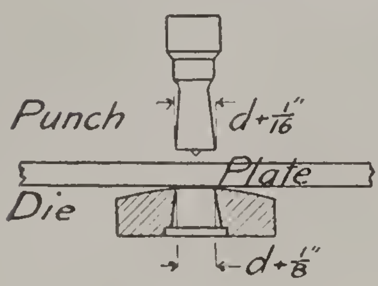


Fig. 3.

Where metal is used of greater thickness than the diameter of the rivets, it is usually drilled. Also some specifications require that all holes shall be drilled in certain cases.

The common practice is to use *"punched work"* for buildings and ordinary highway bridge work, with both soft and medium steel. For railway bridges the practice differs very much on different roads. Usually soft steel over 5/8 in. thick, and all medium steel is required to be reamed. It is probable that the development of manufacture will be toward drilling all holes, which would assure a *fit* and *matching of parts* which cannot be attained with punched work. Even if it were possible to do punching accurately, the matching of holes would be difficult to attain because punching causes a piece to stretch, and the amount of stretch depends upon the thickness of the metal and the number of holes. For this reason, as shown in Fig. 4, it is necessary to ream all holes after the pieces are assembled, so

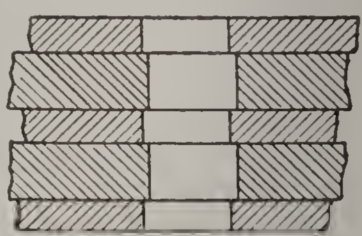


Fig. 4.

[1]All references are made to the 1907 Edition.

that the rivets may be entered. This is usually done with a hand pneumatic reamer. *This reaming does not produce "reamed" work,* as only part of the injured metal around the hole is removed. Forcing round tapered pins, called *"drift pins,"* into the holes with sledges, instead of this reaming, *is not allowable* because it injures the metal.

3. Driving Rivets. Rivets are driven hot, and may be driven in three ways; by power riveting machines, by pneumatic hand hammers, or by hand. Wherever it is practical, rivets are driven by power riveters, because these produce *better results* at a *less cost.*

Power riveting machines are of two kinds, direct and indirect acting. The direct acting are the most satisfactory, as the full pressure may be held on the rivet as long as desired. In these machines the ram moves in the line of the final pressure throughout the stroke. In the indirect acting machines, the cups are held in jaws which are pivoted in the middle, the power being applied at one end of the arm and the rivet driven at the other. This causes the cup to rotate in the arc of a circle. Consequently the cups must be changed every time the grip of the rivet changes or else a lop-sided rivet head would result.

Machines using compressed air are the commonest, and are called *air riveters.* They are used as portable machines, being hung from cranes running on overhead tracks in the shop, and do very good work if of proper capacity and if the air pressure is sufficient. A shop doing girder work should have a machine which will exert a pressure of from 50 to 60 tons. Hydraulic machines are the simplest and most reliable, but they must be used as stationary machines. Steam machines are also stationary, and these machines are therefore better adapted to riveting light pieces than large and heavy ones.

Rivets which must be driven in the field are usually driven by hand, but on large jobs power is sometimes used. There are generally a few shop rivets in every structure which cannot be driven by machine without taking the piece back, after an intermediate operation, like planing, has been performed. Such rivets are also usually driven by hand or with pneumatic hand hammers.

The use of the *pneumatic hammer reduces* hand riveting to

very small proportions. This hammer strikes very rapidly. The blows are comparatively light ones, but very good rivets can be driven with it. Pneumatic hammers are often used for field riveting.

Rivets driven by power riveters or pneumatic hammers, through several thicknesses of plates, which are then planed off to the center of the rivet, will show so tight that it is difficult to see the line of demarkation between the rivet and the plates.

In hand riveting the end of the shank is hammered with hand hammers until it is upset roughly into the form of a head. A *"snap,"* which is a hammer with a cup shaped face, is then held over it and struck with a sledge until the head is properly formed and the rivet is tight. The rivet is held in place while being driven by a *"dolly,"* which is a steel bar with a cup shaped face which fits over the head of the rivet.

The heating of rivets is important, because overheating, or "burning" is injurious, as well as doing work on them at a "blue heat." The range of temperature at which wrought iron may be worked without injury, is greater than for steel, and therefore some specifications require that field rivets shall be of wrought iron. If a rivet is not properly heated it is almost certain to be a bad one. There are also times in a shop, when there is an unusual demand for power, and the pressure used for driving the rivets runs too low.

A loose rivet may be discovered by striking the rivet head a sharp blow with a light hammer specially made for the purpose. An experienced inspector can detect loose rivets by the jar on the hand and the sound produced, even when no movement can be seen. Sometimes attempts are made to deceive inspectors by caulking the heads of loose rivets or by giving them several sharp blows with the riveting machine.

4. Theory of Riveting. In spite of their importance, there is no rational working theory for designing riveted joints and connections under stress. Therefore certain assumptions are usually made, which ordinarily render the design of riveted connections a very simple matter.

In a steam boiler or standpipe[1] the important point is to

[1]For the design of standpipes, see Johnson's "*Modern Framed Structures*," Chapter XXVII.

get a maximum efficiency of the joint, which requires that we have the same factor of safety in the net sections as in the rivets. This subject will not be considered here. In steel buildings and bridges it is simply a question of having, at any point, a sufficient number of rivets and a sufficient net area to take care of the stress at the point.

The following assumptions are made in designing riveted joints:

1. That all rivets completely fill the holes into which they are driven.

2. That the rivets in a compression member take the place of the metal punched out, but that in a tension member the section is weakened because the net section through the rivet holes is less than the gross section.

3. That a rivet cannot safely carry a tensile stress, that is a stress pulling against its head.

4. That the friction between the parts joined should be neglected.

5. That the bending stress in the rivets may be neglected.

6. That the net section of a piece of steel will offer the same resistance *per square inch* as the gross section.

7. That the stress is equally distributed over the net section of the pieces joined in tension.

8. That the stress is equally distributed over all the rivets of a joint.

These assumptions are largely interdependent and will be considered in detail.

If a rivet were perfectly driven, and the hole completely filled when the rivet was hot, it would contract in diameter in cooling. This contraction *precludes an intimate contact between the rivet and the walls of the hole.*[1]

Regardless of this fact *it is the universal practice* to proportion compression members for gross section, and tension members for net section. An allowance should, however, be made in compression members, for *open holes,* or holes for loose fitting

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[1]According to experiments by M. Considere in 1886, the space between the rivet and the side of the hole, varies from 0.002 to 0.02 inches. See Bulletin No. 62 American Railway Engineering and Maintenance of Way Association, page 149.

bolts or pins. The allowance to be made in tension members will be treated in Art. 11.

Coincident with the contraction in diameter while cooling, the length between heads tends to decrease, and a tensile stress is set up in the rivet. In addition to this stress, the metal which is being riveted together is compressed by the enormous pressure exerted by the riveting machine, and when this pressure is relieved, the metal tends to resume its unstrained form, and exerts a tensile stress on the rivet. This initial tension tends further to reduce the diameter of the cold rivet and cause a greater clearance between the rivet and the walls of the hole. The amount of the initial tensile stress on the rivet is a very uncertain quantity. *It sometimes requires a very little pull on the head of a rivet to break it off.* This is probably in part due to the heat treatment which it has received, making it non homogeneous. Nearly all specifications prohibit the use of rivets in direct tension, but they are nevertheless so used in certain connections, because the construction is usual and simple. In these connections there are usually stresses acting at right angles to each other, such as a shearing and a tensile stress. Bolts might be used to take the tension and rivets to take the shear, but rivets are generally used throughout.

Experiments indicate that the clearance between the rivet and the walls of the hole, allows a slip to take place when the friction between the parts is overcome.[1] Therefore *friction is the resisting force in a riveted joint,* so long as the stress is not great enough to produce slip. With good riveting and ordinary working stresses there is probably no slip,[2] nevertheless rivets are calculated to resist shearing off. If a proper working stress is used, the shearing strength of a rivet is a proper measure of the friction produced, because the friction depends upon the tension in the rivet, and that, as well as the shearing strength, depends upon the area of the cross section. In good work the

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[1] See Johnson's "Materials of Construction," Article 375, also pages 3 and 4 of Bulletin No. 62 American Railway Engineering and Maintenance of Way Association.

[2] Experiments indicate that slip occurs at a stress of from 11500 lbs. to 21900 lbs. per sq. in. of rivet cross section.

slip is so small that a joint may safely be strained beyond the slipping point, if the stresses *do not alternate in direction.*[1]

Practically, it is considered of great importance, that the rivets should completely fill the holes into which they are driven. Since this is impossible it is not of so much importance so long as *sufficient friction* is produced between the parts joined. As it requires great pressure to make a hot rivet fill the hole, especially when the holes in the parts joined do not come exactly opposite to each other, (see Fig. 4) this pressure is useful in bringing the parts into intimate contact, which *is necessary* to develop the friction.

If no slip occurs, *the only bending stress in a rivet is due to elastic deformation,* if any at all occurs. The longer the rivet the less the bending stress. Usually specifications require that the grip of a rivet shall not exceed from four to five times its diameter, on the supposition that the rivet transmits the stress. This requirement is necessary, because if the grip is great and the number of pieces to be riveted together is large, the pressure exerted by the riveting machine is not sufficient to bring the pieces into intimate contact and thus develop the friction.

When rivet holes are punched, some of the material immediately surrounding the hole is injured, also a riveting machine exerts an enormous pressure on the metal near the rivet, and may overstrain it. These might tend to reduce the permissable unit stress in tension on the net section,[2] but experiments show that where the section is suddenly reduced, as in a notched bar or in a section through rivet holes, the ultimate strength per square inch is increased by an amount which will more than equal the reduction due to injury.[3]

If then the distribution of stress over the net section through the rivet holes is uniform, as per the 7th assumption, *there is no reason why the allowable intensity of stress should not be as great as for a section without rivet holes.* If, however,

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[1]See Bulletin No. 62 Am. Ry. Eng. & M. of W. Assoc., pages 3 & 4.

[2]See Proceedings of the Institute of Mechanical Engineers, August 1887, page 326.

[3]See Proceedings of the Inst. of Mech. Eng., October, 1888, also see Heller's "Stresses in Structures," Art. 13.

the stress is unequally distributed, the maximum intensity will be greater than the 7th assumption will give.

There are a number of causes producing non-uniform distribution of stress over the net section through rivet holes. If two plates in tension be joined by several rows of rivets, and there is no slip, the stress is transmitted from one to the other by means of the friction at their surfaces of contact. This friction is greatest under the rivet heads, because the friction is produced by the tension in the rivets. Therefore the intensity of stress is greater under the rivet heads than half way between them. If the stress is tensile in the plates joined, the uniform distribution of stress will be interfered with, as in a notched bar.[1]

The result is, no doubt, a somewhat greater intensity of stress near the rivet holes than half way between them.

If the stress is not equal on all the rivets in a cross section, as per the 8th assumption, there may be a large variation in intensity of stress over the section. On this account *the rivets in a joint should be symmetrically disposed about the center lines of stress*, and eccentric stresses avoided wherever possible. If any of the rivets are defective, the result may be the same as that of an unsymmetrical distribution.

If the friction which is produced by the rivets is greatest under the rivet heads, the stress is transferred from one plate to the other in a series of increments. The stress in one plate increases, while that in the other decreases. The result is that the intensity of stress in the two plates at a cross section is not equal, and this tends to cause one plate to deform more than the other and thus throw more stress on the rivets at one end of the joint in one plate and upon those at the other end in the other plate. But the plates cannot deform unequally as long as there is no slip, so *there is no reason why there should not be a uniform distribution of stress over the rivets*, as long as they are all in the same condition. This would require perfect workmanship.

---

[1]See Proceedings of the Inst. of Mech. Eng., October, 1888, also see Heller's "Stresses in Structures," Art. 13.

5. Requirements for a Good Riveted Joint. From the discussion in Art. 4 the following conclusions may be drawn: A good riveted joint,

1. *Should be as compact as possible,* in order to render the uniform distribution of stress more certain.

2. *Should not be very large,* because the workmanship cannot be perfect, and there is the greatest danger of uneven distribution of stress in a joint having the largest number of rivets. With part of the rivets in a joint defective there may be eccentric stresses and overstrain, causing a redistribution of stress and probably overstrain in other members.

3. *Should have its rivets arranged symmetrically about the center lines of stress.*

4. *Should have provision for unavoidable eccentric stresses* (see Art. 12).

5. *Should have rivets of good material, properly driven, under uniform conditions.*

6. *Should have a sufficient number of rivets so that there will be no slip if the stresses alternate in direction.*

7. *Should not have rivets in direct tension.*

6. Proper Sizes of Rivets. The usual sizes of rivets, which are seldom departed from in structural work are ½ in., ⅝ in., ¾ in. and ⅞ in. Rivets larger than ⅞ in. in diameter *can not be driven tight by hand,* and in shops, it is not always possible to obtain sufficient power to drive them satisfactorily.

It is a common rule not to use a rivet diameter smaller than the thickness of the thickest plate through which it passes, because, although somewhat thicker plates can be punched, it is often expensive work on account of the breakage of punches. If thicker metal is used it must be drilled, and the result is that metal of greater thickness than about ⅞ in. is avoided.

Tables giving the maximum size of rivet which can be driven in various sizes of structural shapes, and the location of the most desirable rivet center lines or "gages," are found in the handbooks published by the various steel companies.[1]

Generally it is best to use the largest size of rivet allowable in each piece, unless this would result in a number of sizes in

[1]See "Cambria," pages 52, 53, 54 and 314.

one member, which would cause extra handling in the shop. Usually but one or two different diameters of rivets are used in an entire structure. When two different diameters of rivets are used in one member, the change should be made in such a manner that the two sizes of holes do not both come in any large pieces, as this would necessitate extra handling in punching. Although a 7/8 inch rivet may be driven in a 3 inch leg of an angle, a 3½ inch angle should be used to make an important connection with 7/8 inch rivets.

7. Spacing of Rivets. Rivets are spaced according to practical rules which are almost universal. It is evident that rivet holes might be punched so closely together that the metal between them would be injured to such an extent that it would be of very little value. On the other hand the rivets might be so far apart that the parts joined would not be in close contact between rivets, leaving a space for water and dirt to lodge, causing rust which would buckle the parts and might develop high local stresses. Rivets might also be spaced so near the edge of a piece that the metal would tear out.

By *"pitch" of rivets* is usually meant the distance center to center, *parallel* to the line of stress, whether the rivets be in the same or in different rows. *End distances* are parallel to the line of stress and *side* distances are perpendicular to it. In Fig. 5. p=pitch. e=end distance. s=side distance. d=diameter of rivet. t=thickness of outside plate..

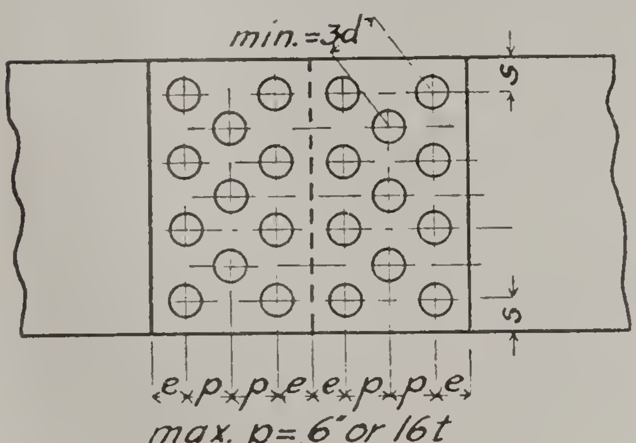

Fig. 5.

The following table gives the usual specified limits for rivet spacing, and Fig. 5 explains the terms used.

Diameter of Rivet in inches *d*	Min. Dist. Cent. to Cent. Specified in inches *3d*	Usual Min. Pitch for Single Line in inches	Maximum Pitch Specified	Usual Maximum Pitch Used	End Distance Specified in in. *2d*	End Distance Usually Used in inches	Side Distance Specified in in. *2d*	Side Distance Usually Used in inches
½	1½	2¼	16*t* or 6 in.	16*t* or 6 in.	1	1¼	1	1 or 1¼
⅝	1⅞	2½			1¼	1¼	1¼	1¼
¾	2¼	2½			1½	1½	1½	1½
⅞	2⅝	3			1¾	1½	1¾	1½

The minimum pitch in a double line may be less than in a single line, so long as the distance center to center of holes in any direction is not less than the minimum distance specified. It is not the usual practice to use the *least* allowable pitch unless there is a good reason for not avoiding it. For the maximum pitch *16t* requires 4 in. for ¼ in. plates and 5 in. for $\frac{5}{16}$ in. plates. It is not good practice to exceed these pitches, but in some classes of work 6 in. is used as the maximum pitch for all thicknesses of plates. In the best classes of work, no metal is used in important parts, less than ⅜ in. thick, in which case 16t=6 in.

The maximum pitch allowed perpendicular to the line of stress is usually about twice that allowed parallel to it, but this is rarely used except in cover plates of compression members, in which case 40t is sometimes allowed.

At the ends of compression members, the pitch is usually 3 in. and should not exceed four times the diameter of the rivet for a distance equal to about twice the depth of the member. This is to insure a uniform distribution of the stress to the several component parts of the member.

The end distance should never be less than 1½ times the diameter of the rivet, and it is usually specified 2 diameters. It should never exceed 8 times the diameter of the rivet.

In the location of rivets it is important to provide clearance for the riveting tool. This has a diameter about ¾ in. greater than the diameter of the head of the rivet, so that from the center of the rivet to the clearance line, the distance should be

at least one-half the diameter of the rivet head plus $\frac{3}{8}$ in. *In special cases* a riveting tool with one side cut off, requiring a clearance but little greater than half the diameter of the rivet head, may be used.

Some shops have multiple punches, which punch a number of holes at one operation, and are usually used in connection with a spacing table. Certain parts are punched on these punches and are not laid out by templet. There are limitations to the spacing which the table can make, and these must be kept in view in making the shop drawings.

Single Riveted.

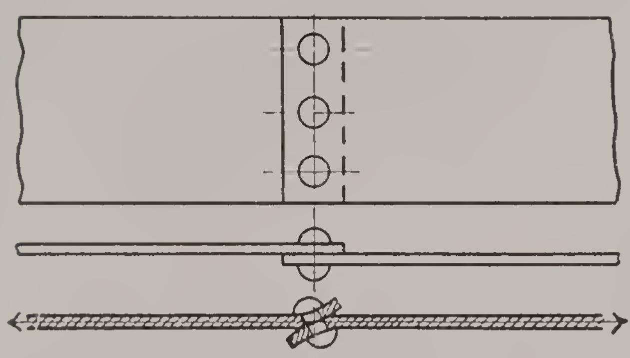

Fig. 6.

Double Riveted.

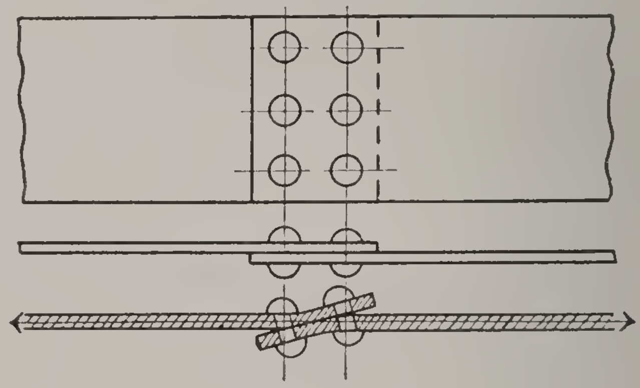

Fig. 7.

LAP JOINTS.

8. Kinds of Joints. Figures 6 to 11 show different kinds of riveted joints in plates, and different arrangements of rivets. It is evident that a lap joint is much weaker than a butt joint with two splice plates. In a lap joint there is a moment, having a lever arm equal to the sum of half the thicknesses of the plates.

Single Riveted

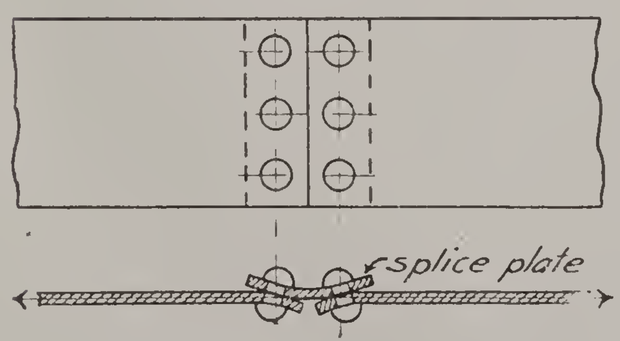

Fig. 8.

Single Riveted.

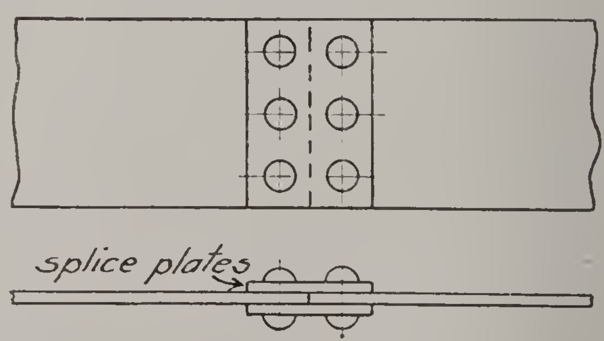

Fig. 9.

BUTT JOINTS

If there were no deformation, the resulting unit bending stress in the plate would be six times the unit stress due to direct stress, but as the joint deforms the center lines of the plates approach each other, as shown in figures 6 and 7, and the moment is

reduced. The bending of the plates will increase the tensile stress in the rivets, and successive changes of stress, if great enough would loosen them.

Butt joints with two splice plates should be used whenever possible.

Staggered Riveting

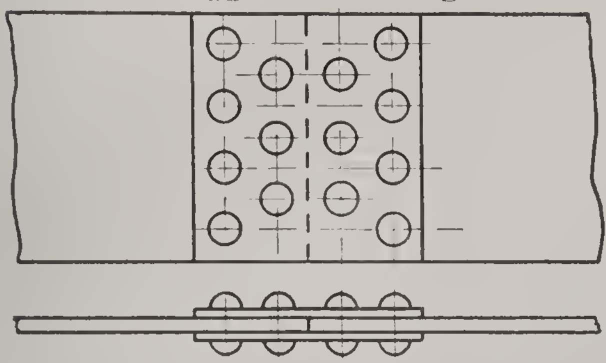

Fig. 10.

Chain Riveting.

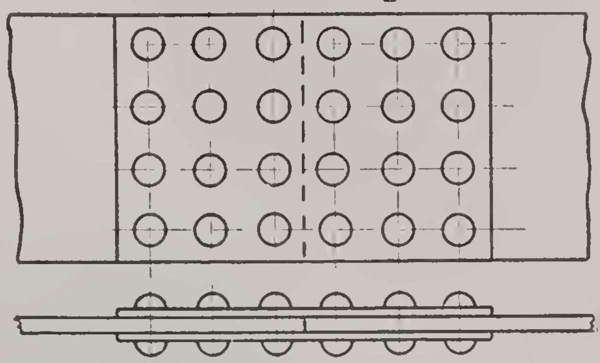

Fig. 11.

9. Design of Riveted Connections. Riveted joints in structural steel work are always designed upon the supposition that the rivets carry the stress according to the assumptions given at the beginning of Art. 4.

According to these assumptions a joint may fail in the following ways:

1. By tearing the parts in tension through a line of rivet holes.
2. By tearing out the metal between the end of the piece and the last rivets.
3. By shearing the rivets on one cross section.
4. By shearing the rivets on two cross sections.
5. By crushing the rivet on one or more of the pieces of metal joined.

Provision against tearing through a line of rivet holes in tension members, will be treated in Art. 11.

The end distances usually specified and which are given in Art. 7, provide against tearing out at the ends.

If there is a tendency to shear off a rivet on one cross section, it is said to be in *single shear*, as in figures 6, 7 and 8. If there is a tendency to shear the rivet on two cross sections, it is said to be in *double shear*, as in figures 9, 10 and 11.

It is evident that if two plates be joined together, one of them might be so thin that the rivet would be crushed where it bears on the plate, before sufficient stress is developed to shear

the rivet off. As the safety against crushing depends upon the area of pressure or bearing, and this depends upon the thickness of the plate, a rivet is said to be in *bearing* on the plate.

Rivets are therefore proportioned for single shear, double shear or bearing. It is possible to have all of these to consider in a single joint.

In a lap joint the rivets are in single shear or bearing, depending on the thickness of the plates. In a butt joint with two splice plates, the rivets are in bearing or double shear. The bearing may be either on the splice plates or on the main plate. The splice plates should always be made thick enough so that the bearing will be on the main plate. That is, each splice plate should be *more than* half as thick as the main plate.

There is no very definite relation, generally recognized, between the working stresses in tension, shear, and bearing. The working stresses for rivets should depend on experiments. Many specifications give a shearing unit equal to about three-fourths of the tension unit and a value in bearing double that in shearing.[1]

The value of a rivet in *single shear* is simply the product of the area of its cross section, by the working unit stress in shear. Thus the area of cross section of a $\frac{3}{4}$ in. rivet is 0.44 sq. in. and its value in *single shear* at a shearing unit of 7,500 lbs. per sq. in. is $7500 \times 0.44 = 3300$ lbs. The value of a rivet in *double shear* is twice its value in single shear. The value of a rivet in *bearing* is taken as the product of the area in bearing by the working unit stress in bearing. The area in bearing is assumed to be the diameter of the rivet multiplied by the thickness of the piece against which it bears. Thus the value of a $\frac{7}{8}$ in. rivet in bearing on a $\frac{1}{2}$ in. plate at 15,000 lbs. per sq. in. is $\frac{1}{2} \times \frac{7}{8} \times 15000 = 6{,}562$ lbs. In designing riveted joints the strength of a rivet is always figured at its diameter before driving. Tables of values of rivets in shear and bearing for several different working stresses, are given in "*Cambria,*" pages 310 and 311.

On account of the inferiority of *field driven* rivets an excess of from 25% to 50% over the requirements for power driven

[1]For experiments on the ultimate resistance of steel and iron plates in bearing see Johnson's "*Materials of Construction,*" Chapt. XXVI.

rivets is usually specified. The frictional resistance is much less with hand driven than with power driven rivets.

The value allowed for rivets with countersunk heads varies with different specifications. If the metal, in which the countersinking is done, is thick enough to give sufficient bearing below the countersunken part to develop single shear in the rivet, no reduction need be made from the value used for rivets with full heads. No reduction is usually made when the heads are only flattened. Rivet heads $\frac{1}{8}$ inch or less high are countersunk.

10. Examples of Riveted Joints. A few examples of the usual forms of riveted joints will now be taken up. In all of the examples in the remainder of this chapter we will assume the following data:

Allowed shearing on rivets 7,500 lbs. per sq. in.

Allowed bearing on rivets 15,000 lbs. per sq. in.

All rivets $\frac{7}{8}$ in. in diameter.

The values of rivets in single shear, double shear, and bearing, may be taken from *"Cambria,"* pages 310 and 311.

Figure 6. The rivets in this single lap joint will transmit $3 \times 4510 = 13{,}530$ lbs. in *single shear,* but if either of the plates be less than $\frac{3}{8}$ in. thick the value of the rivets in bearing on the plate will be less than the single shear value, and the amount of stress which the joint will transmit, will be less than 13,500 lbs. If the plates are $\frac{5}{16}$ in. thick, the rivets will transmit only $3 \times 4102 = 12{,}306$ lbs.

The zigzag line in the table in "Cambria," as explained at the bottom of the page, separates those bearing values which are less from those which are greater than single shear values.

Figure 10. In this butt joint with two splice plates it is evident that the stress must go from the main plate on one side of the splice, to the rivets on that side, from these to the splice plates, from the splice plates to the rivets on the other side, and through them to the other main plate.

The rivets are in *double shear* if the plates are thick enough. From *"Cambria"* we find that the value of a $\frac{7}{8}$ in. rivet in bearing on a $\frac{11}{16}$ in. plate is equal to its value in double shear. If therefore the main plate is $\frac{11}{16}$ in. thick or thicker, and the thickness of each splice plate is sufficient to develop single shear in the rivets ($\frac{3}{8}$ in. or more), the rivets of the joint will transmit

$7\times9020=63{,}140$ lbs. If the splice plates are only $\frac{5}{16}$ in. thick, for example, the rivets will transmit only $14\times4102=57{,}428$ lbs. As stated in Art. 9, the sum of the thicknesses of the splice plates should always be greater than the thickness of the main plate.

Figure 12 shows the lower end of a post which resists, through the pin, the vertical component of the stress in the diagonal tension member. The post consists of two channels 12 in. x 25 lbs. The pin bears against the post in an upward direction, and it is necessary to reinforce the webs of the chan-

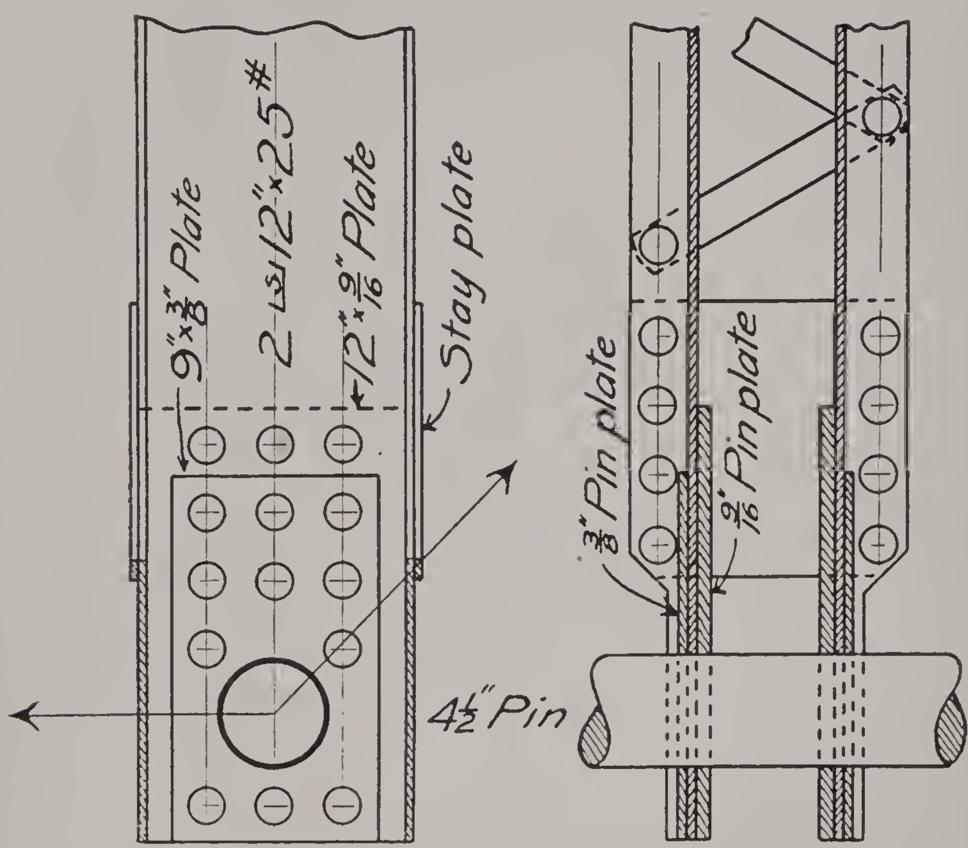

Fig. 12.

nels in order that the pin shall not crush them. Pins are figured in shearing, and bearing, exactly similar to rivets, and usually the same unit stresses are used. Pins must also be figured in bending, and this will be treated in Chapter VII.

Assuming the total stress on the post to be 175,000 lbs., the stress in each channel will be 87,500 lbs. The thickness of bearing on the pin, required to take this stress will be $\frac{87500}{4\frac{1}{2}\times15000}=1.30$ in. The total thickness of pin plates required is 1.30 in. minus the thickness of the channel web, which is 0.39 in. (See "Cambria," p. 164.) The pin plates must then be

1.30—0.39=0.91 in. thick or say $\frac{15}{16}$ in., and may be made up of one $\frac{9}{16}$ in. and one $\frac{3}{8}$ in. plate as shown.

Enough rivets must be put through the pin plates to carry the stresses which they get from the pin to the web of the channel. The total stress 87,500 lbs. from the pin, is distributed over $\frac{21}{16}$ inches thickness of bearing as follows:

$\frac{3}{8}$ in. channel web carries	$\frac{6}{21}\times 87500=25{,}000$ lbs.
$\frac{3}{8}$ in. pin plate carries	$\frac{6}{21}\times 87500=25{,}000$ lbs.
$\frac{9}{16}$ in. pin plate carries	$\frac{9}{21}\times 87500=37{,}500$ lbs.
	Total=87,500 lbs.

There must be enough rivets through each pin plate to transmit its proportion of the stress to the channel web, and there must be enough rivets through the channel web to transmit to it all of the stress from both pin plates.

For that portion of the web which has a pin plate on each side, the rivets will be in bearing on the web, if the web is *not* thick enough to develop double shear in the rivets, and each rivet will transmit from each pin plate, one half the value of a rivet in bearing on the web. Therefore the number of rivets required through the thinner pin plate, will be found by dividing the stress carried by this plate, by one half the bearing value of a rivet on the web.

$\frac{25000}{\frac{1}{2}\times 4920}=11$ rivets required through the $\frac{3}{8}$ in. pin plate.

These 11 rivets will transmit the same amount of stress to the web, from the thicker plate on the other side of the web as from the thinner plate. In addition to these 11 rivets, there will be required through the thicker plate sufficient rivets to transfer the difference between the stresses in the two pin plates, to the web by single shear on the rivets.

37500—25000=12500 lbs. $\frac{12500}{4510}=3$ rivets required through the $\frac{9}{16}$ in. pin plate, in addition to the 11 rivets through both.

It is better to have the pin plates on opposite sides of the web as shown, but if necessary, both plates may be put on the same side, in which case the number of rivets required through

each would be determined by single shear, and these numbers would have to be added together to determine the total number required through the web, as none of the rivet values would be determined by bearing on the web, unless the web were not thick enough to develop single shear in the rivets.

In this example, the rivets below the pin have been counted, but it is evident that they can get no stress except by tension in the pin plates. No more stress can be transmitted to these rivets than can be carried by the net area of the pin plates at the sides of the pin hole.

Also usually, some of the rivets at a joint like this, have to be countersunk on account of clearances, in which case their values must be reduced according to the specifications used as stated in Art. 9.

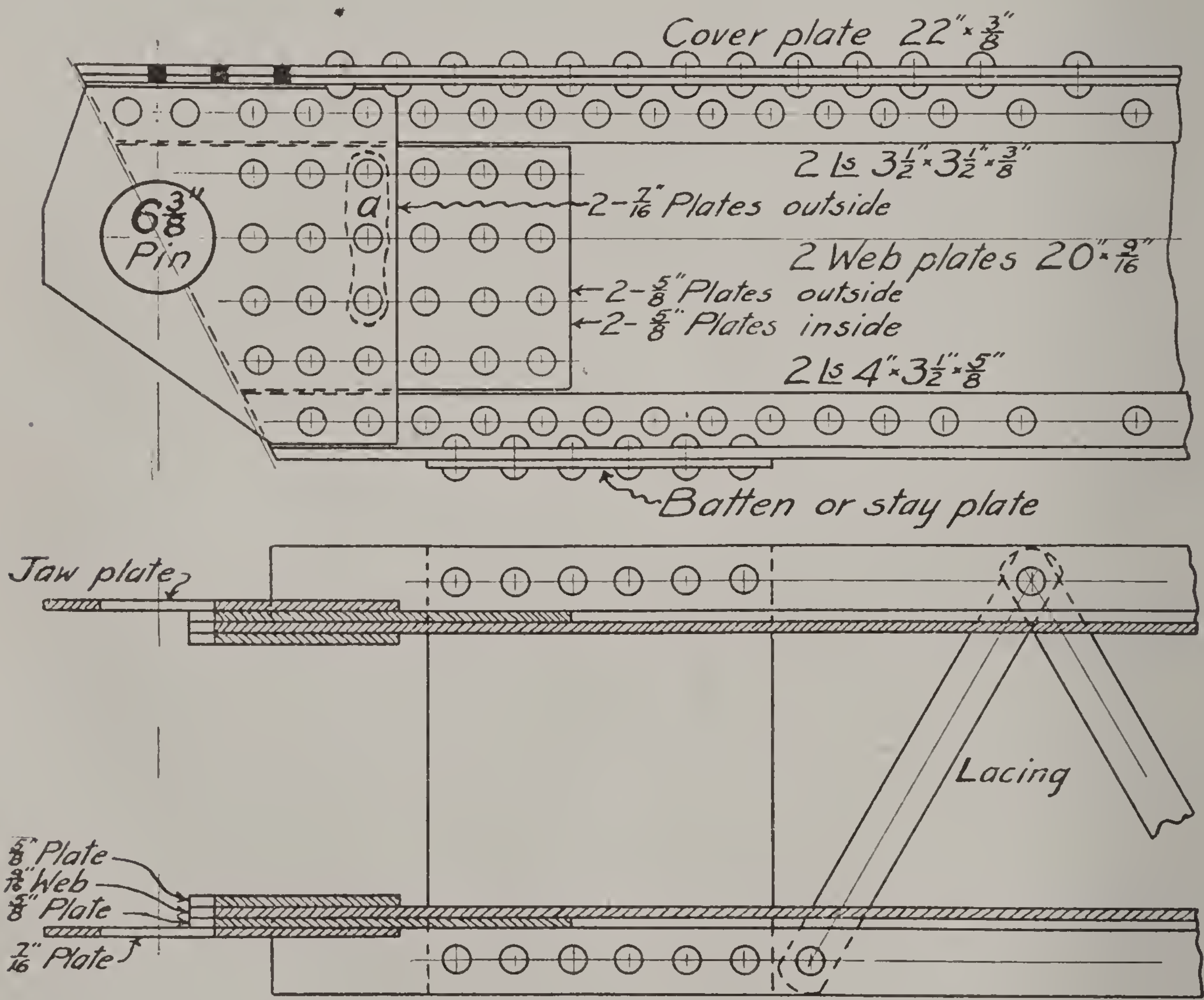

Fig. 13.

Figure 13 shows the top chord of a bridge at the hip joint. The chord section is made up as follows:

1 cover plate, 22 in. $\times$ $\frac{3}{8}$ in.
2 web plates, 20 in. $\times$ $\frac{9}{16}$ in.
2 top angles, $3\frac{1}{2}$ in. $\times$ $3\frac{1}{2}$ in. $\times$ $\frac{3}{8}$ in.
2 bottom angles, 4 in. $\times$ $3\frac{1}{2}$ in. $\times$ $\frac{5}{8}$ in.

We will assume a stress in this chord section of 420,000 lbs., and that the pin is $6\frac{3}{8}$ in. in diameter. This then will require a bearing on the pin of $\frac{420000}{6\frac{3}{8} \times 15000}=4.4$ inches, or say $2\frac{1}{4}$ inches on each side. This bearing thickness may be made up as follows:

Web plate	$\frac{9}{16}$ in.
Inside pin plate	$\frac{5}{8}$ in.
Outside pin plate	$\frac{5}{8}$ in.
Outside pin plate	$\frac{7}{16}$ in.
Total—$2\frac{1}{4}$ in.=	$\frac{36}{16}$ in.

The stress will be distributed over the plates as follows:

$\frac{9}{16}$ in. Web plate,	$\frac{9}{36} \times 210{,}000=52{,}500$ lbs.	
$\frac{5}{8}$ in. inside pin plate,	$\frac{10}{36} \times 210{,}000=58{,}300$ lbs.	
$\frac{5}{8}$ in. outside pin plate	$\frac{10}{36} \times 210{,}000=58{,}300$ lbs.	
$\frac{7}{16}$ in. outside pin plate	$\frac{7}{36} \times 210{,}000=40{,}900$ lbs.	
	Total=210,000 lbs.	

The rivets in that portion of the web, covered by pin plates on both sides, will be in bearing on the web. The web not being thick enough to develop double shear in the rivets. The bearing value of a rivet on the $\frac{9}{16}$ in. web is 7,383 lbs. The number of rivets required in the $\frac{7}{16}$ in. outside pin plate will be $\frac{40900}{\frac{1}{2} \times 7383}=11$ rivets. The number required in the $\frac{5}{8}$ in. inside pin plate will be $\frac{58300}{\frac{1}{2} \times 7383}=16$ rivets. More rivets than required are used in each of these plates, to insure a distribution of stress to the upper and lower rivets, and to give such an arrangement as will put the center of gravity of all the rivets as near as possible to the center line of stress.

The number of rivets through the outside pin plate between the angles, is determined by the single shear value of a rivet and

is equal to $\frac{58300}{4510}=13$ rivets, all of which must be placed beyond the rivets required by the other two pin plates. As there is an excess of three rivets in the two other pin plates, one of the rivets enclosed by the dotted line at "a" may be counted for the outside ⅝ in. pin plate.

Strictly, the rivets passing through the top and bottom angles are in double shear, instead of bearing, because the angles and web are both a part of the main chord section, and are held together by rivets beyond the pin plates. These together make up a thickness more than great enough to develop double shear in the rivets. A filler ¼ in. thick will be required in this case, under the $\frac{7}{16}$ in. outside pin plate, on the top angle. This filler, of course, takes no stress.

One outside pin plate, and all inside pin plates, should take rivets through the angles, in order that the stress may be distributed over he entire chord section, and to the top plate in particular.

This even distribution of the stress requires that the rivets in the ends of compression members, for a distance equal to about twice the depth, should be spaced closely together, as stated in Art. 7.

No pin plate should be shorter than its width, approximately, or it might not be strong enough to carry the stress from the pin to the outer rows of rivets. One of the pin plates should be long enough to extend at least six inches beyond the end of the batten plate.

Here, as in Fig. 12, some of the rivets usually have to be countersunk and their values must be reduced accordingly.

11. Net Sections of Tension Members. In a tension member it is not only necessary to have, in a connection or splice, a sufficient number of rivets, but there must also be a sufficient net area in the parts joined, and in the pieces joining them, to safely carry the stress. Therefore in tension members of a structure with riveted connections, there must be an excess of material, because the joints at their ends, and any splices in them, cannot be made as strong as the body of the member.

Fig. 14 shows a simple tension splice, so made that the net area is as great as possible, and the waste therefore, as small as

possible. If the stress to be transmitted across the joint is 65,000 lbs., it will require $\frac{65000}{6560}=10$ rivets in bearing on the ½ inch plate on each side of the splice.

For getting net areas, the size of the rivet hole is always taken as ⅛ in. larger than the rivet, or 1 in. in diameter for a ⅞ in. rivet. At the section *AB*, the net width of the main plate is 11 in. The net area therefore is $11\times\frac{1}{2}=5.5$ sq. in. If the allowed unit stress in tension is 12,000 lbs. per sq. in., this net area will transmit $5.5\times12{,}000=66{,}000$ lbs. Therefore there is sufficient net area at *AB*.

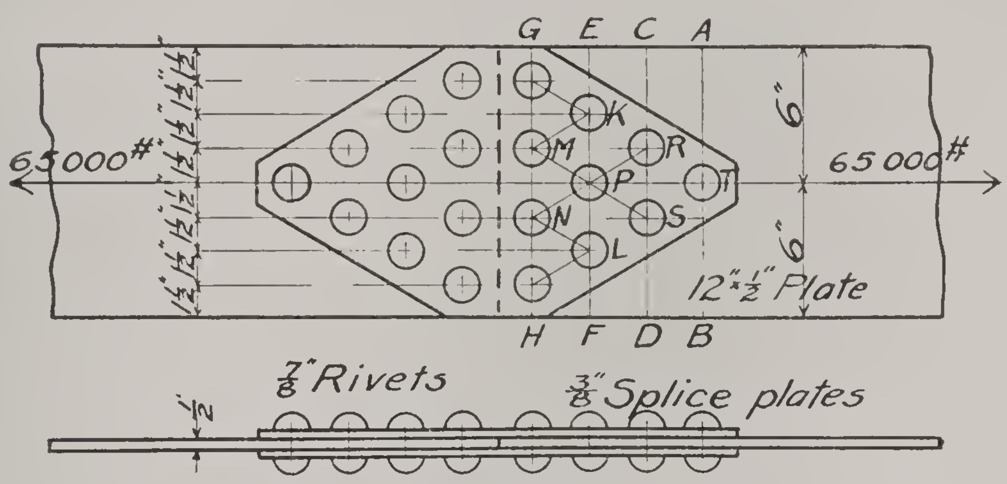

Fig. 14.

At the section *CD* the net area is $10\times\frac{1}{2}=5.0$ sq. in., which at 12,000 lbs. per sq. in. is good for 60,000 lbs. The rivet at *T* has reduced the stress carried by the main ½ in. plate at *CD* to $65{,}000-6{,}560=58{,}440$ lbs. There is then a little more net area on the line *CD* than is necessary to carry the stress.

The stress carried by the main ½ in. plate at *EF* is $65{,}000-3\times6{,}560=45{,}320$ lbs. The net section is $9\times\frac{1}{2}=4.5$ sq. in., which is good for 54,000 lbs. In like manner the stress in the main plate at *GH* is $65{,}000-6\times6{,}560=25{,}640$ lbs., while the net section at *GH* is able to carry $8\times\frac{1}{2}\times12{,}000=48{,}000$ lbs.

The net area of the splice plates at any section must also be sufficient to carry the stress in them, without exceeding the allowed tension unit. At *GH* the stress in the two splice plates is 65,000 lbs. This will require a net area of $\frac{65000}{12000}=5.42$ sq. in. The net width of the plates at the point is $12-4=8$ in., which

will give a required thickness of the two splice plates of $\frac{5.42}{8}$=.68 in. and hence each plate will have to be ⅜ in. thick.

The stress carried by the splice plates at EF is $65{,}000-4\times 6{,}560=38{,}760$ lbs. The required net area is $\frac{38760}{12000}=3.23$ sq. in., which will require a net width of $\frac{3.23}{2\times\frac{3}{8}}=4.31$ in., or a gross width of $4{,}31+3=7.31$ in. The width of the splice plates may be reduced here some, but not in this case to the limit of 7.31 in. because this would not give sufficient edge distance beyond rivets K and L.

In figuring these net areas, only square sections have been taken. It is obvious that if the lines of rivets GH and EF for instance, are close enough together, the zigzag section $GKMPNLH$ will have less net area than the square section GH. Experiments have been made on steel plates which seem to indicate that rupture will take place on the zigzag line unless its area exceeds the area on the square section by at least 30%,[1] and some specificaions require net sections to be figured on this basis. Other experiments seem to show that rupture is equally probable on square or zigzag sections if the net areas are equal.[2] None of these experiments may be a good guide, because there is no doubt an entirely different distribution of stress after the elastic limit is exceeded than before, on account of the unequal deformation and distortion produced.

This is a difficult matter to investigate theoretically, and until further experiments are made, it is well to be liberal in allowances for rivet holes. In Fig. 14 the distance between the rivet lines GH and EF which would be necessary to give 30% excess to the zig zag line $GKLH$ over the square section GH, is nearly 3 in., and if the transverse spacing were greater, this longitudinal distance would also have to be larger.

In nearly all cases in practice, the least area is taken, whether it be zigzag or square section, and no attention is paid to the 30% rule, unless specially required by the specifications.

Some specifications give a simple rule like the following:

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[1]See articles by Prof. A. B. W. Kennedy in Trans. Inst. Mech. Eng., 1881, 1882, 1885 and 1888.

[2]See Engineering News, May 3. 1906, Vol. LV, page 488.

"The number of rivet holes to be allowed for in getting net section shall be the greatest number whose centers are $1\frac{3}{8}$ in.[1] or less from any possible square cross section." According to this rule the rows of rivets would have to be more than $2\frac{3}{4}$ in. apart, if the holes in but one row were to be deducted. This rule is not a safe one to follow in all cases, as will be seen later.

A common case is that of an angle, which may be considered like a plate developed, as in Figs. 15, 16 and 17. The width of the plate will be equal to the sum of the legs of the angle less its thickness. There are four cases according as the angle has one, two, three, or four lines of rivets.

In getting the net area of an angle with one line of rivets, allowance is made for the area cut out by one hole; with two lines for one or two holes; with three lines for one, two, or three holes, and with four lines, for one, two, three or four holes, depending upon the pitch.

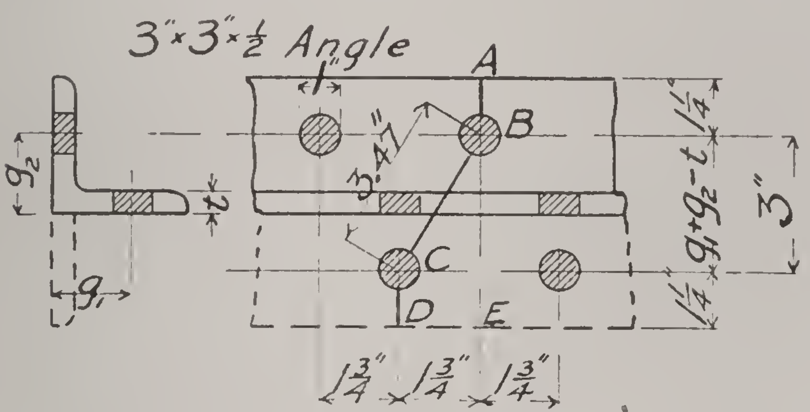


Fig. 15.

In Fig. 15 the stagger of the holes in the two legs is $1\frac{3}{4}$ in., and according to the practical rule above, the net section is equal to the gross section, less the area cut out by two holes, or $2.75-2\times\frac{1}{2}=1.75$ sq. in. It is evident that the holes cut out a large percentage of the material. The square section on AE has an area of $2.75-1\times\frac{1}{2}=2.25$ sq. in., while the zigzag area $ABCD$ is $(3.47-1+2\times\frac{3}{4})\times\frac{1}{2}=1.98$ sq. in., showing a deficiency in place of an excess in the zigzag area.

By working the problem in the other direction we can easily find the stagger of holes necessary to give us either an equal area or a 30% excess on the zigzag line, over the square section. In the case of Fig. 15, it would require the stagger to be at least $4\frac{7}{16}$ in. in order that only one hole need be deducted according to the 30% rule. This would make the holes in one line at least $8\frac{7}{8}$ in. apart.

The following table gives the necessary *stagger* of rivets in

[1]Various specifications give the distance from $1\frac{3}{8}$ in. to $2\frac{1}{4}$ in.

several sizes of angles with one line of rivets in each leg, to give an equal area and 30% excess area on the zigzag section, compared with a square cross section through one hole. By this table we see that the areas given by the practical rule are not always safe.

Size of Angles in Inches	Gage g in inches	Size of Rivet in inches	Area through 1 hole	Stagger for equal area in inches	Area plus 30%	Stagger for 30% excess area in in.
$2 \times 2 \times \frac{1}{4}$	$1\frac{1}{8}$	$\frac{5}{8}$	0.75	1.89	0.98	3.04
$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$	$1\frac{3}{8}$	$\frac{3}{4}$	0.97	2.26	1 26	3.78
$3 \times 3 \times \frac{3}{8}$	$1\frac{3}{4}$	$\frac{7}{8}$	1.73	2.69	2.25	5.53
$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$	2	$\frac{7}{8}$	2.75	2 83	3.57	5.05
$4 \times 4 \times \frac{1}{2}$	$2\frac{1}{4}$	$\frac{7}{8}$	3.25	3 00	4.22	5.68

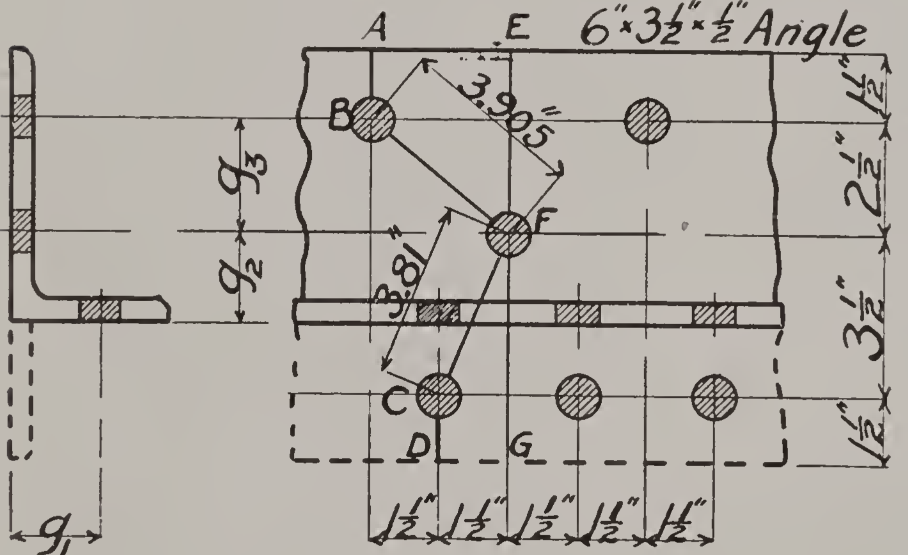


Fig. 16.

In Fig. 16 the area on EG is $4.5 - 1 \times \frac{1}{2} = 4.0$ sq. in. The area on $EFCD$ is $\frac{1}{2} \times (3.81+1.5+4-2) = 3.65$ sq. in. The area on $ABFCD$ is $\frac{1}{2}(3.81+1.5+3.91+1.5-3) = 3.86$ sq. in. Failure would doubtless take place on $EFCD$ or $ABCD$ but according to the 30% rule three holes would have to be deducted.

In Fig. 17 the area on AF is $5.75-1\times\frac{1}{2}=5.25$ sq. in. The area on ACG is $\frac{1}{2}(4.+6.19+1.5-2)=4.84$ sq. in. The area on $ACDE$ is $\frac{1}{2}(1.5+3.9+6.19+1.5-3)=5.04$ sq. in. The area on $ABCDE$ is $\frac{1}{2}(1.5+3.9+3.81+3.9+1.5-4)=5.31$ sq. in. The weakest section is ACG apparently, and failure would probably take place on this section, even though the sections $ABCDE$ and $ACDE$ have far less than 30% excess area over any square section.

If the 30% rule were followed it would be necessary to make

allowance for two holes *at least,* in any angle having holes in both legs, if the maximum allowed pitch were not exceeded. (7)

In order to provide against undiscovered defects in workmanship and material, it is well to make a liberal allowance in calculating net areas, especially where stresses are eccentric, as they usually are in angles.[1]

Practice is not at all uniform on this point.

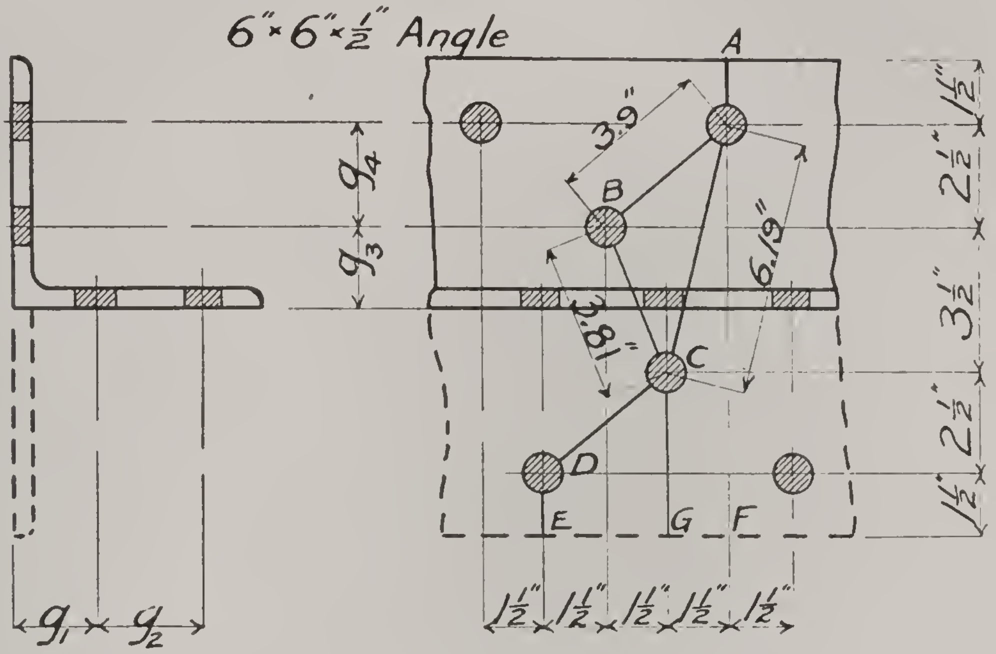


Fig. 17.

12. Eccentric Stresses in Riveted Connections. Eccentric stresses are seldom calculated. They should always be avoided if possible.

It is evident that a single lap joint like Fig. 6 is eccentric. The forces form a couple with a lever arm equal to half the sum of the thicknesses of the plates joined, tending to bend the plates and the rivets (Art. 8). The plates are therefore subjected to a bending and a direct stress. In a butt joint with two splice plates, (Fig. 9), there are no eccentric stresses in the plates joined, but there are in the splice plates.

Figure 18 shows a common form of eccentric connection. The eccentricity might be avoided by moving the force P to P′ so that its line of action will pass through the center of gravity

[1]See Engineering News, Vol. LVI, page 14 (July 5, 1906) for an account of experiments by Prof. Frank P. McKibben, which show that the eccentricity of stress in angles causes rupture to occur at about 80% of the ultimate strength of test pieces cut from the same material.

of the group of resisting rivets. If it is impossible to avoid the eccentricity, the stresses on the rivets may be found as follows.

The force P, of 16,000 lbs., may be replaced by an equal force P' parallel to it, and a couple whose moment is $16000\times 3.18=50{,}800$ in. lbs.[1]

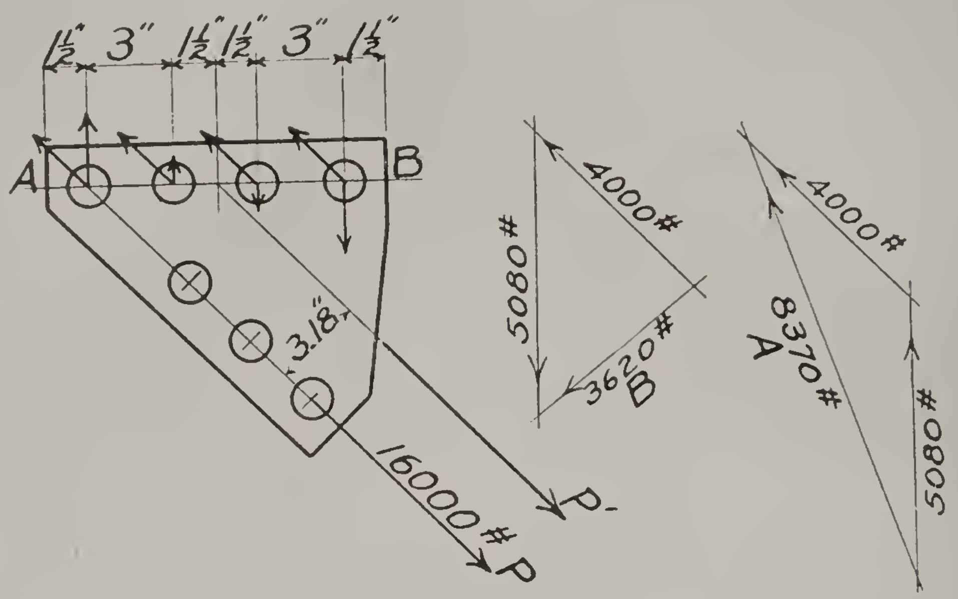


Fig. 18.

The direct stress on each rivet will be $\frac{16000}{4}=4000$ lbs.

To resist rotation about the center of gravity of the group of resisting rivets, *each rivet acts in a direction perpendicular to its lever arm* and thus takes an additional stress *in proportion to its distance from the center of gravity of the group.* If S′ be the stress on each outermost rivet, due to the moment, the equation of moments will be $2\times 4.5S'+2\times 1\frac{1}{2}\frac{1.5}{4.5}S'=50{,}800$ in. lbs., from which we get $S'=5080$ lbs.

The maximum stress on each outer rivet will be the resultant of 4000 lbs. and 5080 lbs., which may be obtained graphically as shown in Fig. 18. The resultant for the two outer rivets will not be the same, because the stresses due to the tendency to rotate act in opposite directions. If the greater of these resultants exceeds the allowed stress on one rivet, more rivets must be used. The resultant for rivet A is 8,370 lbs., and is the greater as would naturally be expected. It is more than double the stress (4000 lbs.) that it would receive if there were no eccentricity.

[1]This is an abstract proposition. See Rankine's "*Applied Mechanics*," Art. 42, also Heller's "*Stresses in Structures*," Art. 34.

Figure 19 shows another common form of eccentric connection. The direct stress on each rivet is $\frac{20000}{8}$=2,500 lbs. The total moment is 20,000$\times$7.42=148,400 in. lbs. Writing the equation of moments we have $4\times7.5S'+4\times6.17\frac{6.17}{7.5}S'=148{,}400$ in. lbs. Solving, we get S′=2,950 lbs., which is the stress on *A*, *B*. *C*, or *D* due to the moment. These stresses act in the directions shown in the figure, which also shows the direct stress on each rivet. Finding graphically, the resultants of the two forces which act on each outer rivet we have for *A* 630 lbs., for *B* 3600 lbs., for *C* 5,410 lbs., for *D* 4,080 lbs. The stress on *C* is more than double what it would be if *P* were applied in the line of P′.

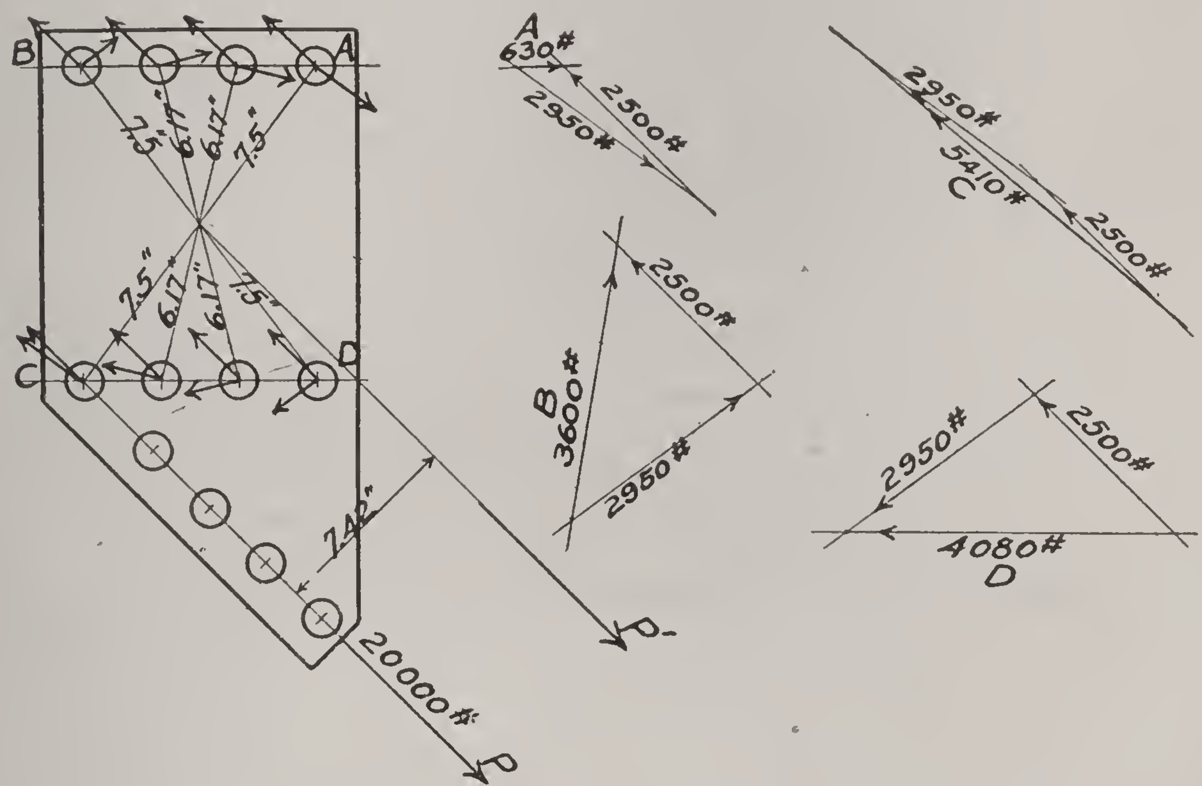


Fig. 19.

It is seen from these examples, that if the connection is eccentric, the rivets are not equally stressed, and that simply taking into account the direct stress will often give results far from the truth.

In laying out a joint in which several members connect, rivet lines are often taken in place of center of gravity lines. This is permissable only when the resulting eccentric stresses come within proper limits. If the rivets in a joint are not symmetrically arranged about the neutral axes of the members,

there will be eccentric stresses. An angle connected by one or both legs forms an eccentric connection which cannot be avoided. (See foot note, page 25.)

13. Showing Rivets on Drawings. In general only rivet heads in plan are shown on drawings. *They should always be drawn to scale.* Where there is any possibility of interference the rivet heads may be shown in elevation as well as in plan. In such cases the heads are sometimes only drawn in pencil to determine the clearance.

In certain locations there is not room enough for a full head, therefore rivet heads may be flattened or countersunk as shown in Fig. 20. By this means heads may be made flush with the metal through which they are driven, or be made ⅛ in., ¼ in. or ⅜ in. high. The usual symbols indicating these various kinds of heads are shown in Fig. 20, which also shows how open holes (into which rivets are to be driven in the field) are indicated. This is called the Osborn system of symbols, and is practically universal in this country now.

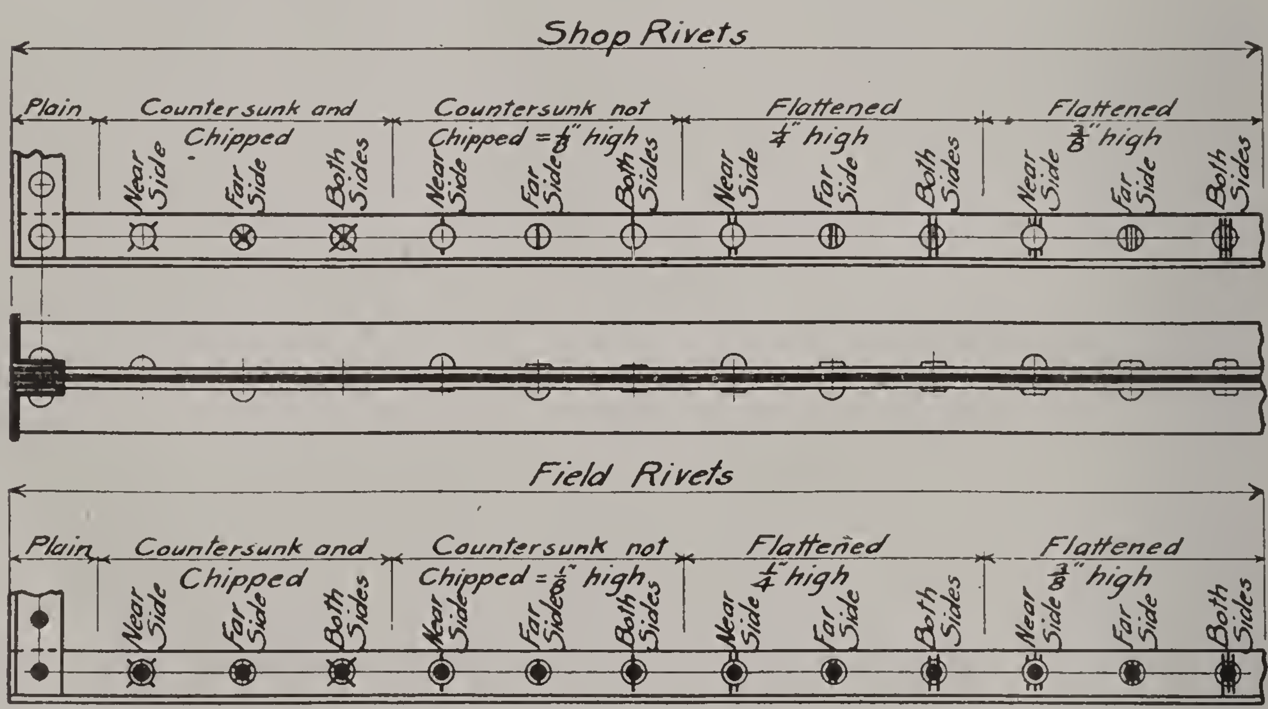


Fig. 20.

All the rivets in a member need not be shown on a drawing, but all of the rivets at the joints should be drawn in. The intermediate portions of the members are frequently omitted and the spacing indicated as so many spaces at so much. In this case the spacing so given should tie up two definitely fixed points at the ends.

CHAPTER II.

DESIGNING AND ESTIMATING.

14. Classes of Structural Steel Work. Ordinarily the term "structural steel" covers only the rolled steel used in structures, and does not include any castings or machinery; but in many classes of work, machinery is so intimately connected with the structure as to render the separate design of the two impossible.

The field of usefulness of steel in structural work is being constantly extended, and the problems of its design becoming more complex, especially for work in the more populus districts of the country.

The following is a list of the more important kinds of structural steel work:

Bridges for steam and electric railways and highways,
- I-Beam spans,
- Longitudinal trough floor spans,
- Through and deck plate girder spans,
- Combination bridges (wood and steel),
- Simple truss spans,
- Draw-bridges (swing, lift, rolling, bascule, etc.),
- Viaducts or trestles,
- Elevated railways,
- Arch bridges,
- Suspension bridges,

Turntables for locomotives,
Trainsheds,
Steel mill and factory buildings,
Steel roof trusses,
Grandstands,
Steel work for tall office buildings,
Stand pipes and elevated tanks and towers,
Steel canal lock gates,
Traveling crane girders,
Ore conveyor bridges,
Car unloaders,
Bins for ore, coal, coke, grain, etc.

15. Kinds of Shops. It may be said that no single plant in this country is well equipped for turning out all of the different kinds of structures enumerated in the preceding article. Some are confined to the manufacture of railway bridges, heavy highway bridges, and heavy building work, some to highway bridges and light building work, some to steel work for buildings, some shops are not equipped to make pin connected work and others cannot do girder work economically. Some shops are not fitted to handle reamed work. (2).

These facts are sometimes emphasized by the manufacturer in order that he may be allowed to make his own designs, but this should not be given too much weight by the purchaser, as all the usual forms of details can be executed in any shop fitted for the particular kind of work under consideration and there will seldom be any difference in price to the purchaser, unless the form of detail is an unusual one.

16. Proposals and Contracts. The requirements of various purchasers in regard to proposals and contracts are not at all uniform. The law requires that public officials advertise for proposals on public work, and any manufacturer who meets the requirements must be allowed to bid. The laws differ in the various states. Private corporations, companies and individuals do not usually advertise for bids, but invite proposals from such manufacturers as they desire to compete for the work. They very seldom require the deposit of a certified check with the proposal to insure the signing of a contract by the successful bidder, or the furnishing of a bond to insure the fulfillment of the contract. The certified check and bond are usually required on public work.

If the purchaser does not furnish a design or plans of the work, each manufacturer submits his own design with his proposal. This may be in accordance with specifications of his own or with some standard specifications. The letting of the contract then becomes a question not only of the lowest bid but also of the most desirable design. Usually the manufacturer submits only a stress and section sheet, commonly called a strain sheet, but sometimes ''show'' plans, showing the general appearance of the structure and some details of construction are also submitted with the strain sheet. Show plans are frequently nothing

more than ornamental drawings on which the lettering and shade lines play an important part.

Railroads have bridge engineering departments which prepare the plans for the bridges. These, together with standard specifications, are submitted to the bidders who then all bid upon the same thing.

17. Designs and Estimates. When an improvement is contemplated the purchaser should employ some one who is competent to prepare plans and specifications and estimates of cost of the proposed work. Also, before making a proposal, the manufacturer must make an estimate of cost, and if no plans are submitted by the purchaser on which to base the proposal, he must also prepare a design and plans to accompany his bid. In either case the method of procedure in making the designs and estimates will be essentially the same. In the case of the manufacturer this work is done by the estimating department.

Designs and estimates must frequently be prepared upon the shortest notice, and in any case must be *completed* before the time set for receiving bids. Certain methods of doing the work of the estimating deparment are of the highest importance, as they save time and reduce the liability of errors.

Proposals for bridge work are asked for either "lump sum" or per pound. In case a lump sum price is required a very careful estimate is necessary. Usually when a pound price is given, only an approximate estimate is made to give a general idea of the various quantities of materials involved.

The estimate of cost includes such items as the following:

Material,
  Steel from the mill (various shapes take different prices).

Eye-bars, Castings, Buckle plates, Hand railing, etc.	In case these are not manufactured by the bidder in question.

Labor of manufacture,
  Shop labor,
  Drafting,
  General expense,
Freight, Haul,

Erection (staging and false work) Painting,
Lumber,
Sub-contract work as paving, masonry work, etc.

Before the cost estimate can be made, of course the various quantities of material required must be determined.

The data furnished for making the design, are frequently very meager but usually include specifications, profiles and maps of the location.

Before starting on a design and estimate, the first thing the designer has to do is to familiarize himself with the requirements and conditions to be met. Of these requirements the specification is often the only one of importance, although in some cases other matters may demand more study. *The specification* is a guide giving the kind of material to use, the loads to be assumed as acting on the structure, the unit stresses allowed, the kinds of details desired, the quality of the workmanship, etc. These will be discussed more fully in Art. 20.

If the *design* is to go in competition with others it is important that it be an *economical* one, that is, the weight must generally be as small as possible and still meet the requirements of the specifications. This is of course important to the purchaser in any case. Designing of structures is not such an exact science that it may be said that all material in excess of what is required to take the calculated stresses is wasted, but the lightest structure is generally the cheapest and usually the *price* is the most important determining factor in selecting a design. Each case must, however, be treated according to the peculiar conditions surrounding it. *Only engineers of experience can design really economic structures.* This matter is an important one because it is a matter of producing a structure to perform a certain function safely and at a minimum cost.

No two estimating departments use exactly the same methods. The following will give the essential points.

In order that the important requirements of the specification may be easily found they may be underscored on the first reading, or an *abstract* made omitting such parts as are common to all specifications and such parts as do not affect the design. If a blank form is used for this abstract it will be better for reference. Calculations should be kept in some permanent form

for future reference, the name of the structure and the date being prominently indicated.

FORM No. 1

THE OHIO STATE BRIDGE COMPANY

Sheet No.____ Made by______________ Date________________

Estimate for____

Span Extreme_______ Roadway__________ Sidewalk__________ Capacity Trusses______ Capacity Floor_______ Specifications	Span C. to C.__________ Panels at__________ Depth C. to C.__________ Length of Diag.__________ Sec. Tg.	Estimated DL per ft. { Steel__________ Floor & Track______ Total_____ Panel Load per Truss DL_______ " " " " LL_______	Total Steel_____ Steel per ft._____ Total Lumber____

(This space for Diagram.)

MEM	DL Stress	LL Stress	Impact	Total Stress	Unit Stress	Req. Area	MATERIAL	Actual Area	No. Pcs.	Wt. P. Ft.	Length	WEIGHT

FORM No. 1. The Form shown in Fig. 22 is to be used in connection with this one.

Fig. 21.

FORM No. 2

THE OHIO STATE BRIDGE COMPANY

Sheet No.____ Made by______________ Date________________

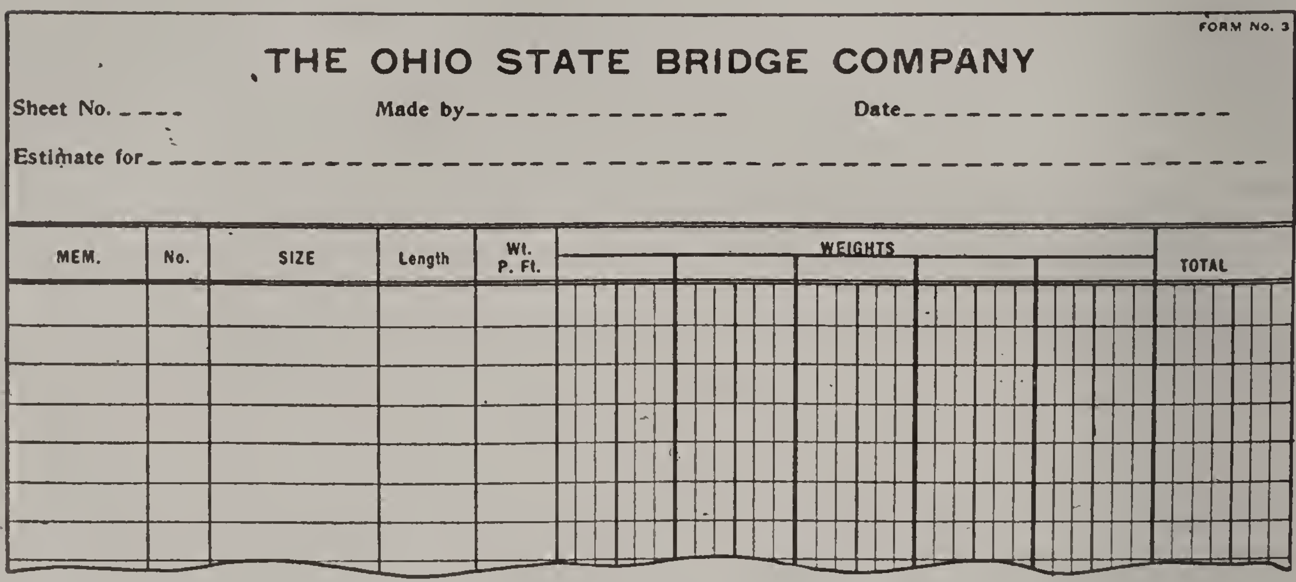
Estimate for____

MEM	DL Stress	LL Stress	Impact	Total Stress	Unit Stress	Req. Area	MATERIAL	Actual Area	No. Pcs.	Wt. P. Ft.	Length	WEIGHT

FORM No. 2. This Form is used in connection with Form No. 1.

Fig. 22.

The information used in making estimates of weight and cost, and stress and section sheets, is set down on blank forms called estimate sheets. If the estimate is from plans giving more or less detail, a form like Fig. 23, may be used. If a design is made in connection with the estimate, blanks like Figs. 21 and 22 are used. The usual size of these is 8½x14 inches.


FORM NO. 3

THE OHIO STATE BRIDGE COMPANY

Sheet No. ____ Made by ____________ Date ______________

Estimate for ____

MEM.	No.	SIZE	Length	Wt. P. Ft.	WEIGHTS					TOTAL

Form No. 3.

Fig. 23.

On Form No. 1, in the space at the top, should be shown a single line diagram of the structure, properly lettered. (For example see Art. 52.) This form also has places for the principal data upon which the design is based, stresses, make up and areas of members and their estimated weights. This form is used for sheet No. 1 of the estimate. For the following sheets Form No. 2 is used, which is similar to the lower part of Form No. 1.

An estimate should give within a few percent, the actual quantities of the various materials, which will be required to make the structure. An estimator must, therefore, not only know how to obtain the weights of main members, but he must be thoroughly familiar with detailing.

Unless the estimate is made from plans giving details, or is for a plate girder bridge, or some such simple structure, the details are not all set down, but are lumped as a percentage of the main parts. A convenient way of doing this, for pin connected trusses, is to make the details of each member, a percentage of the rest of the member, and for riveted trusses, a

percentage of the balance of the whole truss. Of course these percentages will vary with the specifications and form of truss, and must be determined by making an estimate of the details, or by taking them from previous estimates in which the details have been estimated.

An estimating department accumulates, in time, many valuable tables, such as tables of standard connections, joists, rivet and pin values, portal stresses, properties of columns, moment tables, etc., which save much time. Much valuable information is given in the handbooks published by mills rolling structural steel shapes. Those gotten out by the Carnegie Steel Co. and the Cambria Steel Co. are the most complete. Since they give the properties of all rolled shapes, one of them is indispensable in making designs and estimates. Combinations of shapes are frequently used for compression members and girders. Since it is necessary to know the radius of gyration or moment of inertia of these and the location of the neutral axes, and since calculating these involves considerable labor, there should be some systematic method of making the calculations and of preserving the results for future reference. There are several sets of tables published, giving the properties of builtup sections, and one of these will be a great help in designing.[1]

18. Time Savers. Besides the books and tables above referred to there are several instruments, the use of which will save much time and mental effort and reduce mistakes to a minimum.

The most important of these is the *slide rule.* It is, in fact, indispensable in this sort of work. *Thacher's cylindrical slide rule* or the *Fuller rule* are more accurate than is necessary for the ordinary work of the estimating department. A rule which will give results with a maximum error of one in two hundred is sufficiently accurate for all ordinary purposes. The Thacher rule is, of course, very convenient when more refined work is desired. It will give results with a maximum error of about one in ten thousand.

The ordinary ten inch *Manheim rule* will answer very well, but one which will give the product of three numbers at one

[1] "*Osborn's Tables*," by Frank C. Osborn.
"*Properties of Steel Sections*," by John C. Sample.

setting is very convenient. In getting weights, the number of pieces, the weight per foot and the length are multiplied together. There are several rules on which this operation can be performed at one setting. The "*Duplex*" rule is one. This rule is ten inches long and the setting is made on one face and the result read on the other by means of a runner. On the "*Engineer's Slide Rule*" the entire operation is performed on one face at one setting. This rule is twenty-four inches long and has no runner. The great advantage of this rule, aside from its three multiple feature, is the ease with which it may be read. There are only a few more divisions in the twenty-four inches than are given on the other rules in ten inches, and consequently the continued use of the rule is not nearly so trying on the eyes. The degree of accuracy is not much greater than that of the ten inch rules. The maximum error of operations on the three multiple face is about one in two hundred. On the other face of the rule is an ordinary slide rule with scales twenty-four inches long, which gives results within one in five hundred with the same ease on the eyes.

Care should be taken to select a rule which works easily but not loosely, and one in which the graduations on the slide correspond with those on the rule. The trial of a few simple numbers will be a sufficient test of its accuracy. For instance, when the 1 and 2 are set opposite the following multiples should also read *exactly* opposite: 2 to 4, 2.5 to 5, 3 to 6, etc. A rule with a white celluloid face is preferable.

The scales on the slide rule being logarithmic, problems involving multiplication, division, powers and roots can be solved by its use. The books of instruction, which accompany the rules, explain their use fully, but the method of operation can easily be discovered by trial with simple problems. Some definite method of operation should be adopted and always followed, to save time, so that it will be unnecessary to reason out the process each time a multiplication is performed. Most problems may be resolved into the simple form of $\frac{ax}{b}$=*Ans.*, and for these the following simple rule is convenient: "*Keep the DIVISOR on the SLIDE and read the ANSWER on the OUTSIDE.*" Of course, any one of the three factors may be unity, which provides for simple multiplication and division.

The decimal point is best located by inspection after the result has been set down.

If many estimates are made in one office, it should be provided with some kind of an *adding machine*. There are several such machines manufactured in this and other countries. An elaborate machine, such as is used in banks and clearing houses, is not necessary. The "Computometer" is an excellent machine but is somewhat expensive. A small instrument like the "Rapid Computer" is not so expensive and will answer very well.

19. Order of Estimating. It is, of course, very important that all multiplications and additions be correct within certain limits, and every check possible should be employed. *It is also very important that no omissions are made.* The best way to insure reliable results and at the same time secure speed in estimating, is to follow some fixed order of performing the work and some definite form of setting it down.

The following forms have been used by the author and will serve as illustrations:

ORDER OF ESTIMATING RAILWAY BRIDGES

Truss Memb.-Web Diag.
 Web Vert.
 Bot. Chord
 Top Chord
Total Truss Memb.
Pins, Pin Nuts.
Shoes & Mas. Pls.
Rollers & Frames.
Int. Floor Bms.
End Floor Bms.
Int. Stringers.
End Stringers.
Stringer Pedestals.
Stringer Cross Frames.
Stringer Laterals.
Bottom Laterals.
Top Laterals.
Portals—Rods—Knees.
Top Struts.
Sub Struts.
Sway Rods—Knees.
Top & Bot.Struts-Deck Br.
Bot. End Struts.
Longitudinal Struts.
Castings; Lead.
Bolts & Spikes.
Total Iron & Steel.

Timber ft. B. M.
 Furnished by.
 Placed by.
Tie Plates.
Specifications.
Paint—Shop, Field.
Material.
Reaming.
Inspection by.
Transportation.
Haul.
Removal.
Erection.
Bid f. o. b. or Erected.
Certified Check.
Bond.
Penalty.
Time of Completion.
Bid Due.
Substructure.

ORDER OF ESTIMATING HIGHWAY BRIDGES.

Truss Memb.-Web Diag.
  Web Vert.
  Bot. Chord
  Top Chord
Total Truss Memb.
Pins, Pin Nuts.
Shoes & Mas. Pls.
Rollers & Frames.
End Fl.Bms. } Hanger Pls.
Int.Fl.Bms. } Lat.Con.&c.
Sidewalk Brackets.
Fl. Bm. Hangers.
Bot. Laterals } Connections
Top Laterals } Pins &c.
Portals—Rods—Knees.
Top Struts.
Sub Struts.
Sway Rods—Knees.
Fl.Bm.Knees—Low Trusses
Top & Bot.Struts-Deck Br.
Bot. End Struts.
Longitudinal Struts.
Steel Joist—I Bms.
  Facia, Curbs,
  St. Ry., Exp. Joints
Castings.
Bolts & Spikes.
Total Iron & Steel.

Lumber ft. B. M.
Buckle Pls.
St.Ry.Rails-furnished by
  Laid by.
Hand Railing.
C. I. Newel Posts.
Cresting, Ornaments.
Latticed Hub Guard.
Wood Fence.
Paving-Roadways & S. W.
Specifications.
Paint—Shop, Field.
Material.
Reaming.
Inspection by.
Freight.
Haul.
Removal.
Erection.
Bid f. o. b. or Erected.
Certified Check.
Bond.
Penalty.
Payment.
Time of Completion.
Bid Due.
Substructure.

ORDER OF ESTIMATING STEEL BUILDINGS.

Trusses—Main, Vent., Knees.
Lean-to.
Special.
Hip & Valley Rafters, etc.
Columns—Main.
Clearstory.
Crane.
Lean-to.
End.
Floor.
Struts—Latticed Eave.
Side, End, Vent.
Special.
Ties—Bottom Chord.
Special.
Ventillator Knees.
Roof Purlins—Main, Vent.
Lean-to, Special.
Purlins—Side.
End, Gable.
Finish Angles—Main Roof.
Vent. Roof.
Lean-to Roof.
Rods—Rafter, Main, Vent., Lean-to.
Bottom Chord.
Side Sways.
End Sways.
Sag Ties.
Anchor Bolts—Bolts, &c.
Crane Girders—Brackets.
Floor Girders—Joist.
Floor Plate.
Stairs—Tracks for Doors, &c.
Total Steel in Bldg.

Crane Rails-Clips & Fastenings
Corrugated Iron—Roof.
Sides.
Ridge Cap—Flashing.
Gutters—Down spouts.
Slate, Felt, Tin, Cornice, &c.
Louvers.
Wood Purlins, Nailing Strips.
Sheeting ft. B. M.
Skylights—No. & size—Glazing
Windows—No. & size—Glazing.
Doors.
Door Frames—Window Frames
Skylight Frames.
Railings.
Circular Ventillators.
Brick Walls & Foundations.
Specifications.
Materials.
Paint—Shop, Field.
Freight.
Haul.
Removal.
Erection.
Bid f. o. b. or Erected.
Certified Check.
Bond.
Penalty.
Time of Completion.
Bid Due.

20. Specifications. A specification is a set of rules for the guidance of the designer, the draftsman, the rolling mill, the shop, the erector and the inspector. It is a part of the contract[1] and all work is gotten out in accordance with some specification, and for bridges, generally in accordance with some standard specification. There are a number of bridge specifications which are published in pamphlet form for general use.

The following are some of the most used specifications for steel railway bridges:

"General Specifications for Steel Railroad Bridges" of the American Railway Engineering and Maintenance of Way Association.

"General Specifications for Steel Railroad Bridges and Viaducts," by Theo. Cooper.

"General Specifications for Railway Bridges, by Edwin Thacher.

"General Specifications for Railway Bridge Superstructures," by The Osborn Engineering Company.

"General Specifications governing the Designing of Steel Railroad Bridges and Viaducts," by J. A. L. Waddell.

The general specifications for highway bridges usually include specifications for electric railway bridges because bridges frequently serve both purposes. The following are some of the more important of these:

"General Specifications for Steel Highway and Electric Railway Bridges and Viaducts," by Theo. Cooper.

"General Specifications for Highway Bridges," by Edwin Thacher.

"General Specifications for Highway Bridge Superstructures," by The Osborn Engineering Company.

"General Specifications governing the Designing of Highway Bridges and Viaducts," by J. A. L. Waddell.

Most railroads have standard specifications of their own. Manufacturers also have specifications which they use when no other is designated. The *"Manufacturers' Standard Specifica-*

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[1]See "*Engineering Contracts and Specifications,*" by J. B. Johnson, for a complete discussion of the subject.

tions,'' given in the various rolling mill hand books, covers only the material as rolled.

Specifications for bridges carrying electric railways, adopted by the Massachusetts Railroad Commission, have been written by Prof. Geo. F. Swain.

The building codes of the various large cities are supposed to govern the design and erection of all buildings within their limits, but many of these are antequated and cannot be applied to modern types of construction.

''General Specifications for Steel Roofs and Buildings,'' by Chas. Evan Fowler, refers only to mill building construction.

There are many points of similarity in all specifications, especially with regard to certain details. The tendency in the future will, no doubt, be towards more uniformity in all requirements for structures of the same kind. There is no more profitable study for the beginner, than the study of a number of standard specifications.[1] They give, among other things, the types of bridges to be used for different spans, clearances required, construction of the floor, loads to be used in calculating stresses of all kinds, unit stresses which must not be exceeded, details of construction such as lacing for compression members and rollers for expansion bearings, kind of workmanship required, quality of steel and timber to be used, requirements as to painting, inspection, testing, etc.

21. Stress Sheets and General Plans. These are made on tracing cloth and of some standard size. Each company usually has at least two standard sizes of drawings. The common sizes are 8½ in. x 14 in., 11½ in. x 18 in., and 24 in. x 36 in.

The *stress sheet* should show, on a single line diagram, stresses, make up of each member and its area, principal dimensions such as span length, panel lengths, depth and width, and complete general data. Live and dead load stresses should be given separately if the unit stresses are different. The maximum shear and maximum moment should be given for plate girders. It is also well to specify the pitch of rivets in the

[1]For a comparison of the main features of a number of railway bridge specifications, see an article by Prof. A. H. Heller in "Engineering News," Vol. 50, page 444.

flanges of girders, and the number of rivets required in the end connections of floor beams and stringers. (See stress sheets Figs. 35, 53 and 166.)

Full general data should be given for reference in examining the structure after it has been in service for some time and when it may be overloaded. Under this head are included the specifications governing the design, the kind of steel, the location of the structure, the live and dead loads assumed, the grade, the alignment, the skew (if any), the construction of the floor, the distance from masonry to bridge seat, etc.

For bridges, the diagrams usually include an elevation, a half or full view of the upper and lower lateral systems, an end elevation and a cross section.

General plans may be simply "show" plans or plans giving more or less detail. The latter sometimes show practically everything except the rivet spacing and lengths of details.

CHAPTER III.

MANUFACTURE AND ERECTION.

22. Shop Operations. Before taking up the subject of shop drawings, we will consider, briefly, the method of proceedure in the shop work and the erection. This description will be general, as all classes of work are not handled alike and various plants differ somewhat in their equipment and methods.

When the shop drawings on a contract are complete, blue prints of them and the accompanying bills of material are sent to the various departments of the shop. In the *templet shop*, a wooden templet is made for each constituent piece of each different member, excepting, of course, such parts as rods, eye bars, pins, rollers, etc. This templet is of the exact length (or half length) of the finished piece, gives bevels and has holes at every point where a rivet hole is to be located. It is to be clamped to the metal for the purpose of laying out the work to be done on each piece. Laying out directly upon the metal is seldom done because of the danger of making errors and ruining the steel for the purpose for which it was intended.

When the steel arrives from the mill it is unloaded at one end of the plant and marked with the contract number and sizes for future identification. Pieces of the same size are piled together and separated from other sizes as far as possible, so that any material can be gotten out easily at any time without handling other material which is not wanted. The unloading is usually done with a crane of some sort which deposits the material in the yard, or at some plants, in the shop.

When enough material on any contract has been received from the mill and that contract is reached on the shop program, the material is run into the shop as it is needed, and usually continues straight through to the opposite end of the plant where the finished product is loaded.

The first operation after the material is run into the shop is to *straighten* it so that the templet may be applied and that all pieces may be laid out accurately. This is done with presses, rolls and sledges. The next operation is *laying out,* that is,

marking the lines on which the material is to be sheared, and with a center punch, which fits closely in the holes in the templet, the position of each rivet hole. Some material is sheared to length first and then laid out. From the shears or laying out skids, the material passes to the *punches* where all holes for rivets are punched. Next the various pieces which are to be riveted together are *assembled* and fitted, putting enough bolts through the rivet holes to hold the pieces in position until the riveting is completed. These bolts are taken out at the riveting machine as the riveting progresses. Before the pieces are assembled, such faces as will be inaccessible after riveting are painted, and before the riveting is done the holes are reamed out to correct inaccuracies in punching, or if *reaming* is required it is done at this time (2). Some pieces require planing, boring, chipping and hand riveting after the power riveting is done.

After all the operations have been performed on a piece, it is run on to a scale and *weighed* by the shipper, who makes out a shipping bill. Having been weighed and inspected to see that it conforms with the drawing it is *painted* and loaded upon cars or stored to go out when wanted.

All bending, forge work, upsetting, etc., are usually done in the blacksmith shop. Turning, planing (except rotary planing) and all machine work are done in the machine shop.

23. Erection. Putting up the work in the field may be a very simple operation or one involving the use of a large plant and considerable risk, depending upon the character of the structure and its location.

Bridges are usually erected on *false work,* which consists of wooden trestles, by means of a *traveler* or *gallows frame,* to which the tackle for hoisting all material into place is fastened. A *gallows frame* consists simply of two wooden posts connected together at their tops by a beam and braces. The posts usually rest upon temporary stringers outside of the line of the girders or trusses. A *traveler* has four legs, at least, braced together longitudinally and transversely allowing room enough under it to erect the bridge inside of it. It runs on wheels so that it may be moved lengthwise of the bridge as the erection progresses.

Generally during the erection of railroad bridges the traffic must not be interfered with, but trains usually reduce their

speed and run slowly over a bridge which is being renewed. The floor system is sometimes put in place before the trusses and blocked up somewhat higher than its final position. The trusses are erected beginning at the center, putting up one half and then moving the traveler back to the center and working toward the other end. Enough bolts are put into the connections, which are to be riveted, to fill about two-thirds of the holes. After everything is connected together, the bridge is "*swung*," that is, the blocking between it and the false work is taken out and it becomes self-supporting. Rivets for connections of tension members of trusses are driven before the bridge is swung and all others after it is swung.

In the designing and detailing of steel structures it is important that the manner of erecting them be constantly kept in mind. Field splices must be placed in the proper positions, connections should be designed with a view to facility in making them under the conditions which obtain in the field; field rivets should be located where they can be easily driven; sufficient clearance must be provided at all joints. All pieces should have plain marks for identification and a good erection diagram, showing all marks, should be made.[1]

24. The Drafting Department. The organization of the drafting department in various companies differs greatly. Usually there is a *chief draftsman* who has general supervision of all work and assigns the work to the various men under him, whom he deems best fitted to get out the drawings for it. Generally a contract is given to a *squad foreman*, who has three or four men working under him and who directs the method of getting out the work, writes the order bills for the material, and sometimes makes some of the more complicated drawings. When the drawings are made and traced they are sent to a "*checker*," who is generally an old experienced draftsman, and he checks every dimension and size given on the drawing and marks such changes as are necessary, in pencil. The drawing is then corrected by the one who made it, and after being accepted and signed by the checker is ready to send to the blueprint room.

[1]For details of tools, tackle, traveler, false work, etc. see Chapter XIII in Du Bois' "*Framed Structures*," by John Sterling Deans, M. Am. Soc. C. E., and Appendix C in Johnson's "*Modern Framed Structures.*"

The man who makes the drawing and the checker are held equally responsible for any errors.

Drawing boards should be used, as it is very inconvenient to have to remove a tracing from a table top in order to make a lay out or to work a short time on another drawing. The drawing boards should be of pine so made that they will not warp or split. They should have one true edge, preferably of hard wood, at the left hand end.

T-squares should have rigid heads and true edges.

The *drawing table* should be large enough to accommodate the drawing board, reference drawings, etc. It should be at least six feet long, and supplied with a drawer for instruments, etc. It should be, preferably, adjustable as to height and slope of top. The *stool* accompanying it should be adjustable for height.

The *lighting* of the drawing room should, of course, be the best possible. If artificial light is used at any time, it should be a diffused light reflected from the ceiling. A comparatively quiet, well ventilated, clean and orderly office will be conducive to good work and little friction. Unfortunately all of these reasonable conditions are not usually obtained.

A suitable *filing system* should be provided for all drawings and other data, preferably in a fire proof vault.

25. A Draftsman's Equipment. Shop drawings are the working drawings used in the shop and give all details. Making shop drawings is the foundation upon which a bridge engineer's future advancement is based. A draftsman makes his own reputation. Conditions have been such in the past that advancement comes to the draftsman about as rapidly as he is able to take advantage of his opportunities. One who makes himself thoroughly acquainted with the theory of everything he does, one who is not afraid of a little work outside of office hours, who carefully studies and considers every piece of work entrusted to him, will not find the work growing monotonous. A reputation for making mistakes is perhaps the worst a draftsman can make for himself. The fact that every drawing is checked should have no influence upon the amount of care bestowed on it. Errors will sometimes pass the checker and are expensive in nearly all cases. Errors are especially liable to occur when

changes are made necessary after a drawing is made, either before or after it is checked. For this reason a draftsman should do everything according to some method. There is a best place to begin on a structure and a most logical order in which to work it up, not only each drawing but every detail. If this is followed very little erasing will have to be done, and everything will be better designed than if one detail is worked out regardless of everything else and afterwards fudged to correspond with other requirements. After changes are made the drawing is usually out of scale, and not drawing to scale is usually conducive to mistakes. *Even rivet heads should be to scale.*

Every drafting room has some peculiar practices of its own, and it is generally the part of wisdom for a new-comer to conform with them as soon as he can find out what they are.

A draftsman's outfit should include the following *tools*:

1st. Triangles, ruling pen, compass, bow pen, bow pencil. dividers large and small, pen knife, pen wiper, scales, oil stone, etc.

2nd. A copy of some rolling-mill's handbook.

3rd. Tables of squares and longarithms of dimensions in feet, inches and fractions.

4th. A copy of the office standards.

5th. A five place table of the natural functions of angles varying by minutes.

6th. A five place logarithmic table of numbers and functions of angles.

7th. A slide rule.

8th. Reference books.

9th. The following which are usually supplied by the office: drawing tables, drawing boards, T-squares, erasers, soapstone, pencils, pens, tacks, tracing cloth, drawing paper, ink, and chalk.

The *drawing instruments* should be of the best quality. Loss of time due to poor instruments is inexcusable. *Triangles* should be transparent and not less than $\frac{1}{16}$ inches thick. A 5 in. or 6 in., 45 degree triangle and an 8 in. or 10 in. 30-60 degree triangle will be found convenient. For some classes of work a quarter pitch triangle (slope 1 in 2) and a small triangle that will fit the standard bevel of the flanges of I-beams and

channels (1 in 6) will save time. The ruling pen should be of a kind that is easily cleaned because the ink used dries rapidly. There are pencil sharpening machines which do very good work. If there is not one conveniently located in the office, a sharp pen knife can be made to do good work in connection with a piece of sand paper or a file for sharpening the lead. A draftsman who wishes to make a workmanlike drawing will not work with a blunt pointed pencil.

The architects' scale is used for all shop drawings. This is a scale of feet and inches. Scales should not be over 6 inches long for detail work and preferably have white celluloid faces. A long scale necessitates moving the T-square and triangles too much. A 12 inch decimal scale for longer dimensions and graphic calculations should also be provided. A triangular architects' scale is usually divided into the following scales per foot: 3-32 in., 1/8 in., 3-16 in., 1/4 in., 3/8 in., 1/2 in., 3/4 in., 1 in., 1 1/2 in., 3 in. and 12 in. or full size. As most of these scales are used infrequently, it is more convenient to have two flat scales covering the more often used scales. One divided into scales of 1/8 in., 1/4 in., 1/2 in. and 1 in. per foot and another divided into 3/8 in., 3/4 in., 1 1/2 in. and 3 in. per foot will answer most purposes. A scale of 1 1/4 in. per foot makes a very nice scale for some classes of work but it is not a standard scale. The scale of 1/4 in. per foot is much used by architects.

A good *pencil eraser,* one that will not "smear," is, of course, necessary. Ink on tracings should be erased with a *rubber ink eraser* although a steel eraser (knife) may be used occasionally. To prevent ink "running" and dirt accumulating on the spot which has been rubbed, the tracing cloth should be rubbed with soapstone. To confine the rubbed surface within the required limits, it is convenient to have a thin metal *erasing shield* with holes and slots of different sizes.

The *ink* should be waterproof india ink of good quality.

For drawing lines on detail paper a *6H pencil* should be used, because it does not require such frequent sharpening as a softer pencil. For putting in dimensions and figures a *4H* pencil will be about right. On tracing cloth a *3H* pencil will work best, and a soft pencil is needed for scratch figuring.

A pen must be "broken in" before good lettering can be

done with it. After it has been used for some time the point becomes blunt and may be improved by using a knife on it as on a pencil in sharpening it.

The *rolling-mill hand books* contain much information of use to the draftsman, and he should know what may be found in them. The pages most frequently referred to should be indexed for quick reference in some manner similar to a ledger index. The principal shapes rolled by the various mills are all alike and in accordance with the standard adopted by the Manufacturers' Association. All properties of these shapes are given in the hand books. The American Bridge Company's Standards give much other valuable information.

Each office usually has a set of *standard tables* and drawings. Many of these are of general value, but some correspond with certain local shop practices.

Tables of squares and logarithms of dimensions in feet and inches are in constant use, and are a much greater help than would appear on first thought.[1] In working with right angle triangles, only the table of squares is necessary. The table of logarithms is not used so often, but when there is use for it it saves much time. These tables give results to the nearest 1-32 of an inch, and this is the smallest fraction ever used on structural steel drawings.

The above tables, together with a good table of the natural functions of angles such as is given in *Trautwein's Pocket-book*, and a five place table of logarithms such as Gauss's or Jones' will enable the draftsman to solve all problems in mensuration which may arise, if he is thoroughly familiar with the fundamentals of geometry and trigonometry.

The draftsman should be familiar with the use of the *slide rule*, and should use it to calculate pins, rivets, bearings, etc. (18) If the office is provided with a *Thacher Cylindrical Rule* it may be used to good advantage in calculating the dimensions

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[1] "*Tables of Squares*," by John L. Hall and "*Buchanan's Tables of Squares*," by E. E. Buchanan, give the squares of dimensions under 50 feet, expressed in feet and inches. Tables by Thos. W. Marshall give the logarithms of the same quantities. "*Smoley's Tables of Squares and Logarithms*," by Constantine Smoley, give both the squares and logarithms of these dimensions in parallel columns.

in oblique triangles, and will give results within 1-32 of an inch so long as none of the lengths involved exceed about 30 feet.

A note book is very convenient for keeping calculations which one may wish to refer to again. Many figures which a draftsman makes will be on scratch paper, but all figures which may be needed for future reference and for consultation when the changes made by the checker are gone over, should be kept in a permanent and methodical form. A careful man will find satisfaction in seeing how he made a mistake, and this record will also be valuable in giving reasons for certain things he has done, and prevent his being misled by some one who, perhaps, has not considered all the conditions.

A draftsman should have at hand *reference books* in order that he may look up any point in theory with which he is not familiar. A few may be mentioned here but, of course, for structures out of the ordinary special works should be consulted. The most useful are:

Johnson's *"Theory and Practice of Modern Framed Structures,"* Heller's *"Stresses in Structures,"* some good work on the strength of materials, Wright's *"The Designing of Draw Spans,"* Freitag's *"Architectural Engineering,"* Merriman and Jacoby's *"Bridge Design"* (Part III of Roofs and Bridges), Kent's *"Mechanical Engineer's Pocketbook,"* Trautwein's *"Civil Engineer's Pocketbook,"* Engineering News, etc. Access to the Transactions of the American Society of Civil Engineers, and of other engineering societies will be valuable. An individual card index should be kept in order that any subject may be looked up when occasion requires.

26. Ordering Material. As soon as a contract has been secured and entered, complete data relating to the construction are turned over to the drafting department. The first thing to be done by this department is to prepare a list of the material required, which is called an *"order bill."* The draftsman to whom this is entrusted should first carefully examine all data. If any necessary information is found lacking at any point in the progress of the work, it should be promptly reported. Care should be taken to include everything in the order bill that will be required to make the structure, unless upon inquiry it is found that certain materials may be ordered later.

Since the order bill must, in general, include all details such as pin plates, batten plates, lacing bars, rollers, pins, eye bars, rods, timber, lead, corrugated iron, windows, doors, crane runway rails, etc., it is necessary to proportion all details, to calculate pins, rivets, bearings and connections, and to determine clearances, splices, etc. In some cases considerable drafting will be required, but not nearly so much as will be necessary to make complete detail drawings. In general, this preliminary drafting should be done so that after the order bill is complete the drawings may be developed into final shop drawings. This method will save much drafting, which is expensive work.

In order to expediate the placing of the orders for the material at the mills, no more drafting than necessary should be done. It may be supplemented by free hand sketches and notes, which should be preserved with other data for reference by the checker and draftsman making the shop drawings.

For a contract of any magnitude the order bills would be gotten out in sections; those parts which will be needed first should be gotten out first. In any case, the first attention should be given to the kind of material which will be most difficult to get promptly.

However, time spent upon a consideration of the entire contract in all its bearings, and especially with regard to duplication of parts, is never wasted.

The order bills are checked and a copy sent to the order department. The originals are retained for the guidance of the draftsmen who make the shop drawings. The element of time is so important that most companies do not make blue prints of the order bills but use some more rapid duplicating process.

The order department makes up a *"mill order"* from the order bills, bringing together all items of the same kind and combining some of the shorter lengths into long lengths (multiple lengths) to be sheared to the proper lengths at the shop.

Some companies keep more or less material in *stock* in order to be able to make prompt deliveries of certain classes of work. In this way considerable waste (short pieces) accumulates, which must be applied on contracts whenever an opportunity offers. It is the duty of the order department to keep track of the stock, to keep up the supply of stock sizes, and not

to allow an accumulation of waste. When stock material is applied to a contract it should be so marked, in order that it may be reserved. The *shop bill* generally shows what items are to come from the mill and what items from stock. Stock material cannot always be used on contracts, as some specifications require a different quality of material from that carried in stock and require inspection at the mills.

The importance of avoiding *errors* and *omissions* in the order bills is apparent, since they may cause serious delay to the entire work.

In making up the order bill, it should be remembered that odd sizes of angles and other shapes should be avoided in order to get prompt delivery from the mill. $\frac{1}{4}$ inch extra material should be ordered for all tool finished (milled or planed) surfaces, except for the flat surfaces of plates for which 1-16 in. or $\frac{1}{8}$ in. extra should be ordered, depending upon the size of the plate. Stiffener angles which are to be crimped or offset, are usually ordered as long as the depth back to back of angles of the girder to which they belong. The length of bent angles and plates is taken on the center of gravity line. Other material is ordered the exact length required (avoiding smaller fractions than eighths of an inch) and usually comes a little longer so that it may be sheared to the finished length.

Plates are of two kinds, "*sheared*" and "*universal mill.*" The former have sheared edges and the latter rolled edges. The maximum width of universal mill plates varies from 20 in. to 48 in., depending upon the mill where they are rolled. In the rolling mill handbooks will be found tables showing the maximum length to which plates of various widths and thicknesses can be rolled. Plates up to 7 in. in width are called *bars*. Flange plates for plate girders are usually obtained as long as wanted, but web plates must be spliced. It is usual to order web plates of a width $\frac{1}{4}$ in. less than the depth back to back of angles, and to allow $\frac{1}{4}$ in. clearance between their ends. It is permissable to order odd shaped plates sheared to the dimensions wanted, as shown by a sketch.

Angles may be obtained in single pieces up to about 90 feet long, provided that they do not weigh more than about 3,000 lbs. apiece. Special sizes should be avoided on account of slow de-

livery. The order bill should indicate the kind of material (soft or medium) and the specifications governing its quality and inspection.

27. Shop Drawings. Drawings should be made on the dull side of *tracing linen,* because they will not lie flat when made on the smooth side. An experienced draftsman will work directly on the tracing cloth and this cannot be done except on the dull side. Drawings may be fastened to the board with thumb tacks, but it will be found more satisfactory to use very small carpet tacks, tacking the four edges like a carpet so that the drawing will be stretched and present as smooth a surface as possible. Changes in temperature and moisture of the air may sometimes necessitate restretching. The drawing should be covered up at night with a heavy cloth or a piece of table oil cloth. If the drawing is made on the tracing linen direct, the principal lines may be inked before the drawing is completed in pencil. This will bring out the main parts and make it more satisfactory to work with, especially if there is much work on the drawing, as the lines are liable to become faint from rubbing over them. It is, however, safer for an inexperienced man to make a complete pencil drawing before doing any inking.

In order that the tracing cloth may *"take"* the ink it is necessary to rub it with pulverized chalk, Fuller's earth, or blotting paper. This is especially necessary during cold and damp weather.

The draftsman should not get the idea that the *appearance* of a shop drawing is of no importance. A drawing should have a workmanlike appearance or it will not inspire confidence in its correctness. The general arrangement and the lettering are the main features so far as appearance is concerned. All *lettering* should be free hand and the draftsman should, at the beginning, practice with exceeding patience some simple style of lettering.

The style used in the drawings published in the Engineering News is a very good one to follow. A very important point is to have the letters and figures of different sizes, depending upon their importance. The sizes of materials should be more prominent than the rivet spacing, and center lengths than secondary dimensions. There is also a best position for each dimension.

The letters and figures should be made as carefully as is consistent with rapidity. It is only practice, persistent and patient, that can make a good letterer. Not all can hope to become equally proficient, but all can *improve.*

The *general appearance* of a drawing depends very much upon the general arrangement, the scale, and the relative sizes of letters. A drawing may cover practically all the available space within the border lines, if there is no evidence of crowding anywhere, and if the various parts or pieces represented stand out clearly so that the different views of the several pieces can not be confused. There is an advantage in compactness, but *clearness is the first consideration.* Not every one who uses a drawing can read it as readily as the man who made it. **HE SHOULD MAKE IT SO PLAIN THAT IT WILL EXPLAIN ITSELF AND THAT ONLY GROSS NEGLIGENCE WILL ALLOW ANYONE TO MAKE A MISTAKE IN USING IT.**

The object should be not to make it "good enough" but to make it *first class.*

The length of a structure, like a truss, seldom determines the *scale* of the drawing. Usually the available width of a sheet determines the scale to be used. If the structure is too long to show on one sheet to this scale two or more sheets may be used. The center line diagrams of trusses are usually drawn to a smaller scale than the details, say ½ in. or ¾ in. per foot while the details would be 1 in. or 1½ in. per foot. In this case, of course, there is a part of the member near the middle which is cut out and which need not be shown, but the spacing of rivets, etc., is indicated. True projections are not always made if they do not serve to make matters clear. Bottom views are seldom made but sections instead, placed below the elevation. The top view should always be above the elevation. A half top view and half section are sometimes made together when symmetry will allow. A single view or two views will sometimes suffice, especially if it is a construction with which the shop is familiar. There should be no more drafting than is necessary for clearness.

Drawings should have one or two plain *border lines.* If two are used the blue prints may be trimmed to the outer one. (See Fig. 24.) In some drafting rooms an outfit for printing *titles*

on tracings is used; in some rubber stamps are used. Printing is the most satisfactory method when the number of drawings turned out is large. Titles should correspond with some standard as indicated in Fig. 24. The title should, when possible, be in the lower right hand corner. When necessary it may be divided in two parts placed side by side. A supplementary form for general information is frequently placed in the lower left hand corner as shown in Fig. 25.

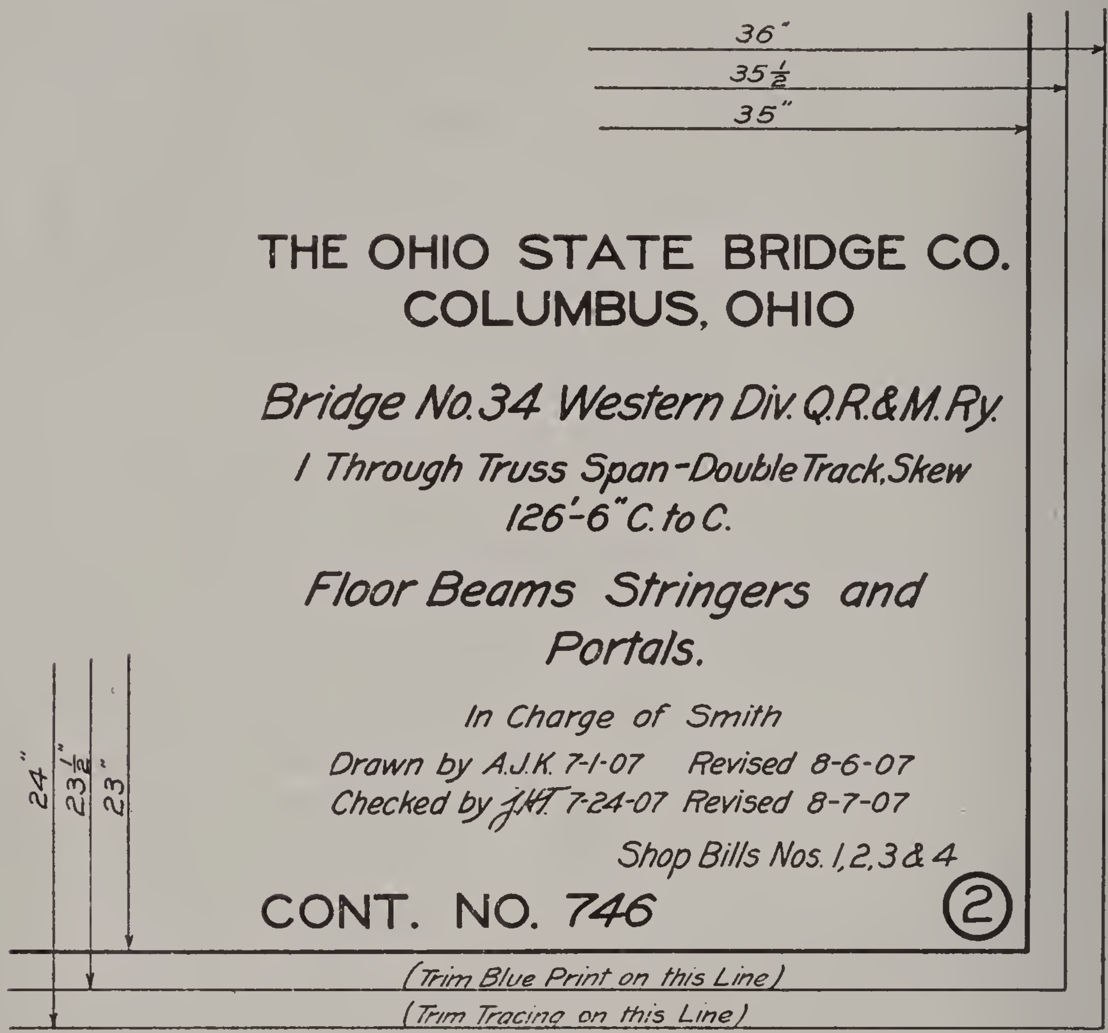

Fig. 24.

Dimension lines and rivet gage lines should be very fine and preferably made with black ink. They are sometimes made with red ink, but the ink should be of known quality in order that it may not run or spread with age. Center lines which form one end of a view should be heavy dot and dash lines; other center lines should be fine lines. No *shading* is attempted on shop drawings except to show curved surfaces.

All lines of dimensions should connect completely with the centers, and there should be separate lines for center distances, rivet spacing, lacing spacing, etc.

Rivets connecting lacing bars to compression members should stagger with those in the web of the member. The end bars should connect to the first rivet in the batten plate or one not over 5 in. from this rivet. Batten plates should be made of such widths as will fit the spacing of the lacing and meet the requirements of the specifications.

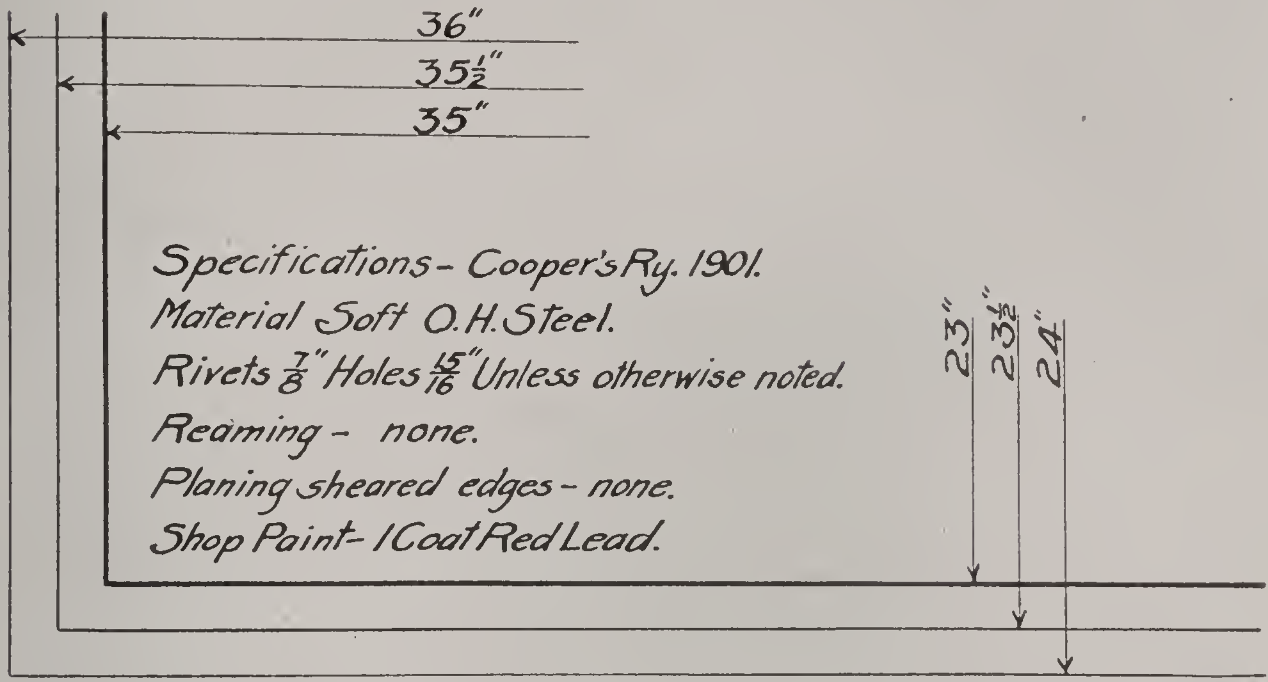

Fig. 25.

Dimensions which determine clearances for field connections, the position of open holes, etc., should be given in such a manner as to be convenient for the inspector. It should not be necessary for him to add rows of figures. If the inside distance at the end of a member is the important one for clearance, that should be given. If the member is to be entered inside of another, its outside width should be given. In some cases both are necessary.

For *identification* in the drawing room, shop and field, each piece should have a mark. All pieces which are *exactly* alike, should have the same mark. The marks should all appear on the general marking diagram, or erection plan, which shows the relative location of all the pieces by their marks.

Under the drawing of each piece the number required should be plainly given together with the numbers which are to be right and left, thus,

2 Right Girders Req. Mark G R (shown)
2 Left Girders Req. Mark G L.

The system of marking, for each kind of structure, should be standard as far as possible. For example, U might always stand for upper, L for lower, P for portal, V for vertical, D for diagonal, B for bracket, S for stringer, F for floor beam, etc. Marks should be as simple as possible, and preferably consist of capital letters and figures, avoiding primes and subscripts. To insure shipment, small pieces which the drawings show bolted to large ones, may be given separate marks and noted on the shipping bill.

Steel in *section* is shown by uniform cross hatching or in black. See Figs. 26, 28, and 29. Other kinds of material are seldom used except for draw-bridge machinery. If some convention is adopted for each kind of material, it will serve to make the drawing clearer.

It is well to follow some *conventions*. If a member is vertical in a structure, it should be drawn with its axis parallel with the sides of the drawing unless this would necessitate the use of too small a scale, in which case it may be drawn parallel with the top of the drawing, with the top of the piece at the right. Inclined members, when not shown in their natural position, should be drawn lengthwise of the sheet.

Notes may be used when they will save considerable drafting, but should generally be avoided. Making the drawing complete will guard against mistakes in the office and the shop. *A note should be so worded that its meaning cannot possibly be mistaken.* It is not permissable to refer to a reference; the drawing referred to should give full information. In any case each drawing should give sizes of all material, pin sizes, center dimensions, and other important information.

Duplication in details, in spacing, in parts of members and in members is very important. The number of templets is by this means reduced to a minimum. It is permissable to use a little extra material to obtain duplication in some cases. When two members differ but slightly from each other, one drawing, with proper notes, will answer for both. If two drawings are necessary and some parts of one are the same as for the other, it is better not to repeat the rivet spacing but to refer to the other drawing, as this will call attention to the fact that there is a duplication of templets. A templet is frequently made to

answer for two different pieces by putting into it all the holes for each piece and marking one set of holes in some way to distinguish them from the other set.

Rivet Spacing should be as regular as possible. All rivet heads need not be shown but they should not be omitted at the ends of members, where clearances are important, in pin plates, or where countersinking or flattening are needed. *Field holes* should always be shown blackened, and it is generally a good thing to show them in at least two views. The conventional signs for countersinking and flattening (See Fig. 20) should be made very plain lest they be confused with dimension lines. No countersinking is allowed in the tension flanges of stringers, floor beams or girders. All countersinking should be avoided in long pieces since it involves an extra shop operation and long pieces are expensive to handle.

All open holes should be so located that the field rivets may be easily driven. Rivets are driven from the sides or from above, never from below. It is not good practice to put two consecutive rivets on the same line in an angle having two gage lines, except for purposes of symmetry, and when it cannot be avoided, as in the connection of a floor beam to a girder.

Punching of holes of different sizes in the same piece should be avoided as much as possible, especially in long pieces, because it requires extra handling. Avoid two or more shearings at the end of an angle or the edge of a plate when one will answer just as well. Projecting corners should, however, not be allowed. Whenever a reentrant cut has to be made, there should be no sharp angle but a curve.

In giving dimensions over 9 inches the feet and inches should generally both be given, thus 0′-11″, 3′-7″. All dimensions over one foot, except the widths of plates, should be given in feet and inches; width of plates are always given in inches, thus, 1--37″×⅜″×3′—6½″. The longer leg of an angle should be given first. Rivet spacing should not be given by repeating a number of consecutive spaces that are just alike, but should be indicated as follows:

8 spaces at 3″=2′—0″
8 alternate spaces at 1′—3″=10′—0″.

Unless the stringers rest on the bottom flange of the floor beam, shelf angles should be provided for them to rest on for convenience in erecting. Where stringers rest on the bottom flanges of the floor beams, and where inside splice angles are used, they should be ground to fit the fillet of the flange angle.

If angle laterals are used, which have lugs riveted to them, it is easiest to make the back of the angle the center line.

It should be remembered that angles are not exactly of the nominal size, but that the length of the legs overruns, except for sizes rolled in finishing rolls. Making fillers and splice plates ¼ inch shorter than the nominal distance between flange angles will not always answer.

Entering connections should be avoided. They make erection expensive and are liable to result in injury to the material.

Wherever two or more members come together, clearance should be allowed if possible. The thickness of an eyebar head, if figuring clearances, is always taken $\frac{1}{16}$ inch greater than the nominal thickness. A total further clearance of ¼ inch to ⅜ inch is allowed where several members enter between the sides of another, the amount depending upon the members so entering, the number of pin plates, etc. Pin fillers are, for the same reason, made ¼ inch shorter than the space they are to fill.

Projecting plates should not be riveted to large pieces, but shipped loose. It is better to drive a few more rivets in the field than to have these details broken off in shipping and handling, or have them interfere with the handling of heavy pieces. Lateral plates for plate girder spans may be riveted to one of the laterials connecting to them if the laterals are not too long.

Care should be exercised so that no part of the lateral system will interfere with the floor construction or the masonry.

It is important to know in what order the spans of a viaduct will be erected and to arrange the details at the tops of the columns so that each span may be put up independently.

Holes for anchor bolts must be so located that the masonry may be drilled after the steel work is in place.

At panel points, not only those rivets which come opposite a member in its final position should be flattened or countersunk for clearance, but also enough to allow the members to be easily assembled. Batten plates should not come too close to a diagonal member.

Top chord splices should come opposite each other in the two trusses, and the sections nearest the center should extend over at least two panel points so that in erection this panel will be self-supporting. It should be remembered that the traveler must be moved and cannot generally support pieces except for about one panel length.

There are two general methods of making shop drawings. *First,* a structure or part of a structure may be drawn showing all parts assembled in their proper relative positions. A bridge, for example may be drawn showing the truss members (usually half of one truss for a square span) in the relative positions in which they belong, while a separate drawing is made of each of the other pieces such as floor beams, portals, etc. This method, of course, is not adapted to some things, like floors and columns of office buildings. *Second,* each kind of piece belonging to a structure is drawn separately and complete in itself. In the case of a truss for example, this necessitates the making of a layout of each joint beforehand, in order to determine clearances, and the fitting together of the parts. This method requires more drafting than the first, and is therefore more expensive. The first method is nearly always used for bridge and roof trusses unless the depth is so great that it would necessitate too small a scale.

At some plants the templet shop is arranged to permit laying out a structure full size. The templet maker locates the rivets which are shown but not exactly located on the drawing. Where this is practiced drawings are made in a somewhat different manner, as to what dimensions are given, than where all details including rivet spacing are shown.

The beginner should always have a sample of the kind of structure of which he is required to make a drawing, for a guide. There are many practical points which can only be picked up in this way. He should also make himself familiar with the machines in the shop and their capacities. It is sometimes as easy to design a masonry plate which will go into the planer as one that is too wide for it.

Before starting on the drawings for any particular structure, a draftsman should make himself perfectly familiar with all the data. Time spent in a general preliminary consideration

and plan of action is generally well spent. If further information is required, it should be asked for *at once.* If a mistake or omission is discovered in the order bill, the attention of the engineer in charge of the office, or of this particular contract, should be called to it *at once.*

Working to the order bill may make some trouble, but this is necessary. Should any doubtful points come up, some one should be consulted who is more familiar with this kind of work, or with the requirements of the parties for whom the work is to be built. *A man should never be ashamed to ask intelligent questions.*

28. Order of Procedure for a Pin Connected Bridge. No definite order of procedure can be outlined, which can be followed in all cases, but the following order for a pin connected truss bridge will serve as a guide to the beginner.

The stress sheet and specifications form a part of the contract and the draftsman must work from these.

1st. Write on a blue print of the stress sheet the horizontal and vertical components of the stresses in the inclined members.

2nd. Determine the location of the centers of gravity of the compression members and decide on the location of the center lines.

3rd. Determine all the center lengths so as to give the required camber.

4th. Make a table of heights as follows:

Depth of tie over the stringers	=	
Depth of the stringers	=	
Bottom of stringer to Bot. of Fl. Bm.	=	
Bottom of Floor Bm. to Pin Cent.	=	
Base of Rail to Pin Cent.	=	(Sum)
Pin Cent. to Masonry	=	
Base of Rail to Masonry	=	(Sum)
Lower Pin Cent. to Base of Rail	=	
Required Clearance	=	
Lower Pin Cent. to Clearance	=	(Sum)
Depth of Truss C. to C. Pins	=	
Depth of Portal, Vert. from P. C.	=	(Diff.)

5th. If the bridge is on a skew, calculate the lengths and bevels necessary to draw the portal.

6th. Calculate the size of masonry plate required so as not to exceed the allowed pressure on the masonry and so as to provide enough room for the rollers.

7th. Assume sizes of pins, determine thicknesses of pin bearings on each member and calculate the pins. When the pins are all calculated decide upon two or three sizes for the truss, making some larger than necessary for the sake of simplicity. Fix on the location and thicknesses of all pin plates and the number of rivets required in each. The calculation of the pins will have determined the packing at each panel point.

8th. Calculate the pitch of rivets required in the flanges of the stringers and floor beams and the number of shop and field rivets for their end connections. (This information is frequently given on the stress sheet.) (21)

9th. Calculate the rivets required in the lateral systems, including portals, to take the longitudinal and transverse components of the stresses as required.

While performing the above preliminary work, a few sketches will be necessary, and it is important to decide upon the form of the connections for the lower lateral system, allowing clearances for eyebar heads, and to see that the steel work will fit the masonry, allowing sufficient clearance at the ends for expansion.

No rigid rule can be laid down for the best order in which the different parts should be drawn up, but for this class of structure the following will work out satisfactorily:

The scale for the center line diagram is usually $\frac{3}{8}$ inch, $\frac{1}{2}$ inch or $\frac{3}{4}$ inch per foot, and for the details, $\frac{3}{4}$ inch, 1 inch, $1\frac{1}{4}$ inch or $1\frac{1}{2}$ inch per foot. The 1 inch scale is used more than any other one for details.

1st. Draw the portal, especially if it is a skew bridge. A layout of the hip joint will be necessary.

2nd. Work out the hip joint with portal connection, lateral connection, pin plates, etc.

3rd. Work out the shoe joint with rollers, anchor bolts, masonry plates, shoes, end strut or beam and lateral connections, pin plates, etc.

4th. Work up the spacing of the lattice bars, batten plates and rivets in the end posts.

5th. Work up the lower chord joints, with floor beam connections, beginning at the middle of the truss and working toward the end.

6th. Finish the hip vertical.

7th. Draw the top chord joints, and fix spacing for lattice bars, batten plates and rivets in top chords.

8th. Finish the intermediate posts.

9th. Finish the top view of top chords.

10th. Draw top lateral system and top struts.

11th. Draw bottom lateral system and end struts.

12th. Draw stringers and beams.

13th. See that each drawing has a proper title and number, and that all general notes required are on the drawings.

14th. Go over each drawing to see that all information which may be wanted by the following persons, is given: The checker, the templet maker, the layer-out, the fitter-up, the inspector, the shipper, and the erector. See that each piece is properly marked and the number wanted is given, that the sizes of rivets and open holes are all given.

15th. Make a marking or erection diagram (single line) and a diagram showing how the bridge is to be located on the masonry.

16th. After the drawings are checked, look into all corrections carefully before doing any erasing. *Do not erase the checker's marks.* In case you do not understand the checker's changes or see any reason for them, ask him for information. The important thing is to have the drawing clear and correct.

Riveted truss bridges can be handled in much the same way as pin connected bridges. The scale for the truss drawing is usually somewhat smaller than for the details. Great care should be exercised in order that connections may be as free from eccentricity and as compact as possible. (12) It is important to so space rivets that the net section of any member will not be less than was contemplated in the design. (11) When a connection is too complicated to exactly proportion the rivets, assumptions should be made which will be on the safe side.

29. Order of Procedure for a Plate Girder Bridge. The method of procedure for a plate girder bridge may be outlined as follows: The scale of the drawing of the girder should be ¾ inch, 1 inch or 1¼ inch per foot. Two or more sheets may be used for long girders. For a skew bridge the full length of one girder must be drawn. The girder should be drawn first, but it will be necessary to sketch the lateral and beam connections before it can be completed. The first thing to be considered is the location of the splices in the web. The hand books give the maximum lengths obtainable for plates of different widths and thicknesses. Web plates up to about 96 inches wide may be obtained longer than convenient for handling in the shop. Their length should be limited to from 20 to 25 feet, except for girders less than 30 feet long whose webs may be made without a splice.

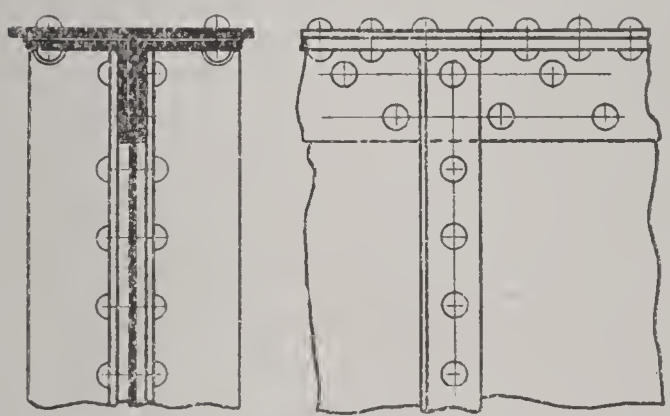

Fig. 26.

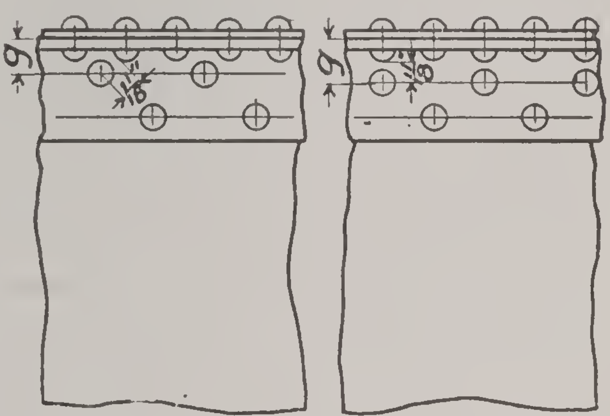

Fig. 27.

The pitch of the rivets in the flanges should be determined and regular groups of spacing be decided on, as for example, 2¼ inch, 2½ inch, 3 inch, 4½ inch and 6 inch. Now the splices may be located. If it is a deck girder, they should be so located that there will be no odd spaces, if possible, in the groups decided on. Stiffeners are always placed on the splice plates. The intermediate stiffeners need not be spaced so exactly equal but that they may come on one of the rivets previously located, except where the pitch in the flanges is less than 3 inches, in which case a space of at least 3 inches will generally be required on each side of the stiffener. If the girder is for a through bridge, the splices will usually be located at the panel points, from which everything else must be located. Through girders often have their top flanges bent to a quadrant at the upper corners.

and extend down the ends of the girder. This necessitates splices in the top flange near the bend so that long pieces will not have to be bent. All splices in a flange should break joints.

In general no countersinking is allowed in girders except in the shoe plates. When the rivets in the horizontal leg of a flange angle stagger with those in the vertical leg, some of the former will interfere with the outstanding legs of the stiffeners. To avoid this, special spacing may be introduced near the stiffener or the stiffeners notched as shown in Fig. 26. In unusual cases it may also be desirable to notch the other leg of the stiffener to clear a rivet.

The horizontal legs of the flange angles are usually as wide or wider than the vertical legs. When the vertical leg is 5 inches or over, two rows of rivets are used in it. The practice with regard to the horizontal leg differs. The simplest way is to have also two lines of rivets in each horizontal leg, putting those of the inner row in one leg opposite those of the outer row in the other leg. This, however, gives more rivets than necessary through the flange plates. If only one row of rivets is used in each horizontal leg, they should stagger with those in the vertical legs. If two rows are used in each leg the spacing in the horizontal legs may be increased to one and one half or two times that in the vertical leg. For example, where the spacing in the vertical leg is 2¼ inches, 3 inches and 4½ inches, that in the horizontal legs might be 4½ inches, 4½ inches and 6 inches, or 4½ inches, 6 inches and 6 inches. When the spacing in one leg is 3 inches, for instance, and that in the other is 4½ inches, the rivets on the inner row of the 4½ inch spacing will stagger with those in the other leg, while those in the outer row will come opposite. Three lines of rivets are sometimes used in 7 inch and 8 inch legs of angles.

The minimum pitch (7) of rivets depends upon the distance between rivet lines and the gage of the latter, and is influenced by clearance for the riveting tool. In Fig. 27 "*g*" depends upon the thickness of the angle. For large angles, therefore, there are usually two standard gages. In crimped stiffeners the distance "*a*" Fig. 28 should not be less than 2 inches. Stiffeners should be placed with the backs of the angles toward the ends of the girder. Flange rivets should not be located closer to

stiffeners than shown in Fig. 29. This allows room enough for the regular die of the riveting machine.

Even if fillers are not required under stiffeners, it is best to use them at points where beams or frames connect and at the splice plates. Connections for beams and frames should, if possible, be made in such a manner that they may be swung into position without striking the rivet heads in the flanges of the girders.

For girder bridges on a grade, the girders should, if possible, be made so that they will fit if turned end for end. The bevel should be in the masonry plates and not in the shoe plates.

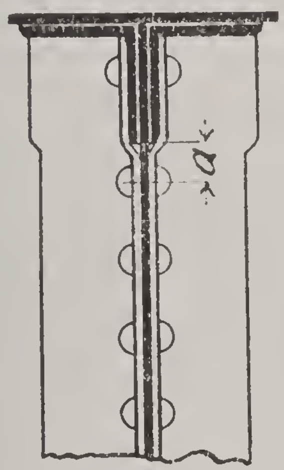

Fig. 28.

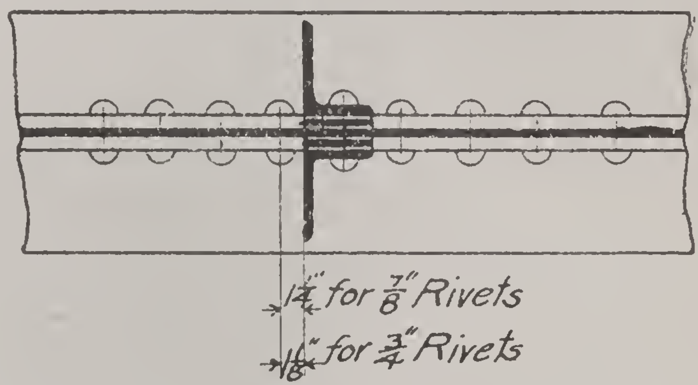

Fig. 29.

30. Shop Bills. Shop bills are lists of material for use in the shop. They are made on sheets 8½x11 inches or 8½x14 inches. The forms used by different companies differ somewhat, but the essential features are as follows:

They should be numbered consecutively, and should show the number of the drawing to which they refer. Each finished piece should be billed separately, and the number to be shipped should appear in the first column. In the second column should be given the number of constituent parts required to make the number of members shown in the first column. The main parts of members should be given first, the details following, putting the lacing last. Shop rivets are not billed.

The size of each part, the name or location, or both, and its length, must be given. Both the finished length and the ordered length should be given in separate columns. In the "remarks" column it is usually indicated whether or not a piece is to come from the mill or from stock. Blacksmith work, machine work, and riveted work are usually put on separate bills. Blank forms are sometimes used for pins, bars, field rivets, etc. Be sure that nothing is omitted, as it might seriously delay erection.

A *check list* for various kinds of structures should be prepared, giving all possible items to ship, similar to the "Order of Estimating" given in Art. 19. By consulting these, omissions may be avoided.

It is usually required to send drawings to the shop or to have them printed for approval, as soon as they are finished. If they are sent for approval it may be better and more convenient not to make the bills until the prints are returned approved, as there may be some changes. Five or six sets of prints are required for the shop. Sending out drawings before all parts of the structure are drawn up is not the most logical thing to do, but is often necessary.

FORM No. 4

Sheet No. 25 Cont. No. 243

THE OHIO STATE BRIDGE COMPANY

Trusses Span No. 3 Date 9-3-04

Bridge No. 16 Q. R. & M. Ry. Drawing No. 14

Ship	No. Pcs.	Kind	SIZE	Wt. P. Ft.	DESCRIPTION	Finished Length	Ordered Length	REMARKS
4	Interm.		Top Chord	Sec.	CD-R & L			
	4	s	25" × 3/8"		Cover Pl.	21'-9 1/16"	21'-9 1/2"	Carnegie
	8	Ls	3 1/2" × 3 1/2" × 3/8"	8.5	Top Ls	21'-9 1/16"	21'-9 1/2"	"
	8	Ls	" × " × 5/8"	13.7	Bot. Ls	21'-9 1/16"	21'-9 1/2"	"
	8	s	16" × 3/8"		Webs	21'-9 1/16"	21'-9 1/2"	"
	8	s	9" × 3/8"		Web splice	1'-0"	12'-0"	M.L. "
	4	s	"		Cover "	2'-1"	12'-0"	M.L. "
	4	s	15" × 3/8"		Batten Pls.	2'-0 3/4"	8'-4"	M.L. "
	88	s	2 1/4" × 3/8"		Lattice 15 1/8" 15 1/8"			Stock
2	Interm.		Top Chord	Sec.	DD			
	2	s	25" × 3/8"		Cover Pl.	24'-9 1/16"	24'-9 1/2"	Carnegie

Fig. 30.

A simple form of shop bill is shown in Fig. 30. The following information should also be put on a bill sheet and sent with the shop bills:

Contract No.	Ship to
Description	Shop Paint
Location	Field Paint
Date to Ship	Lumber furnished by
Inspected by	Penalty
Erected by	

In general, the drawings should show everything complete, but the drawings of the forge work, machine shop work, and miscellaneous details are often made on bill sheets.

31. Shipment. Field splices and connections are often determined by the limitations of transportation facilities. These should be determined at the outset. Some routes can take care of pieces of greater extreme dimensions than others. Tunnels often limit the width of the loading and overhead bridges, the height. Sharp curves have an important bearing on loading long pieces, especially girders. It should be remembered that a piece extending over two or more cars, swings away from the center line of the track on a curve, requiring more clearance than on a straight track, and that a car on a curve is inclined toward the inside of the curve. Ordinarily, pieces about 10 feet high can be transported on the railroads, but the width at the top is usually limited to about 6 feet. The greatest width of a steel car is 10 feet 2 inches. Pieces 10 feet or more in width can be transported if they are not too high or too long. The question of weight is not generally an important one, except that cars of proper capacity must be used so that no truck will be overloaded. It is allowable to put two-thirds of the nominal capacity of a car on one truck.

For *export* shipment special instruction must be obtained on each job as to maximum lengths, weights, etc. Pieces for export work must, in some cases, be so small that they can be transported on pack animals. A thorough and simple system of marking is necessary. Instead of marks, colors are sometimes used.

The question of cost of freight is sometimes an important one, and may determine the maximum length of a piece. If the total weight of a contract is less than a car load, so far as cost of transportation is concerned, no piece should be longer than a car length.

There are two kinds of *freight rates*, "car load" and "less than car load" (C. L. and L. C. L.), the latter being the higher. The minimum car load is generally 30,000 lbs.; the minimum for two cars is 40,000 lbs.; and 20,000 lbs. is added for each additional car. Therefore if any piece extends from one car over part of another, freight must be paid on at least 40,000 lbs., no

matter how much less the shipment weighs. In case of a girder extending over three cars, the minimum amount charged would be on 60,000 lbs.

32. Materials. The materials used by the structural engineer are wrought steel, wrought iron, cast steel, cast iron and timber. *Cast steel* and *cast iron* are used for the machinery of draw bridges. Except in special cases of columns for buildings, and pedestals, and for small details like washers, ornaments and separators, *cast iron has passed out of use entirely* in structural work. *Cast steel* is sometimes used for shoes of bridges.

Timber is used for the compression members in combination bridges and roof trusses, and for the floors of bridges.

Wrought Iron is used for rods which must be welded,—rods with loop eyes, or forked heads. In the best classes of work welds are entirely avoided. Welds in steel are not considered reliable.

The qualities of the materials required are given in the specifications governing the work. We shall consider structural steel more in detail.

At present *three kinds of steel* are commonly specified, viz., "rivet steel," "soft steel" and "medium steel." Of these, rivet steel is the softest and medium steel the hardest, or the steel of greatest ultimate strength. "Hard steel," or steel having an ultimate strength greater than 70,000 lbs. per sq. in., is seldom used for bridges or buildings.

There is at present a movement toward the adoption of a single grade of steel for all structural purposes.[1] This would simplify the steel maker's work, and no doubt result in a more uniform and a cheaper product.

The quality of steel is determined by analysis and test. There is considerable uniformity in the requirements of various specifications. These may be enumerated as follows:

Chemical requirements,—Percentage of phosphorus, sulphur, manganese, silicon, carbon, copper and arsenic.

Physical requirements,—Process of manufacture, uniformity, finish, heat treatment, ultimate strength, elastic limit, elongation, reduction of area at point of fracture, appearance of frac-

[1]See Bulletin No. 62 American Railway Engineering & Maintenance of Way Association.

ture, bending, bending after quenching, punching, drifting of punched holes, variation of cross section and full sized tests.

Chemical analysis determines the amounts of the impurities in steel. All elements except iron and carbon may be called impurities. Since it is practically impossible to eliminate all the phosphorus and sulphur, a small percentage of manganese is considered advantageous. In order to meet the physical requirements the manufacturer must limit the amount of all impurities, and of the carbon. All specifications, however, limit the amount of phosphorus allowed, since this element renders the steel brittle or "cold short" while it, at the same time, hardens it. Some specifications also specify maximum allowable percentages of sulphur, manganese and silicon. The maximum allowable amount of phosphorus in steel depends upon its mode of manufacture. It is usually about 0.04% for basic open hearth steel and 0.08% for acid open hearth and Bessemer steels.

Carbon has the greatest influence upon the ultimate strength or hardness of steel. Phosphorus, manganese and sulphur make steel harder and also reduce its ductility.

Basic open hearth steel will, in general, have about the following ultimate strengths, depending upon the percentage of carbon:

55000 lbs. per sq. in. with 0.10% carbon.
60000 lbs. per sq. in. with 0.15% carbon.
65000 lbs. per sq. in. with 0.20% carbon.
70000 lbs. per sq. in. with 0.25% carbon.

The elastic limit will be about 0.6 of the ultimate tensile strength.

There is no rigid line separating the different grades of steel, but steel having less than 0.15% carbon is generally soft steel, and with more than 0.30% carbon, hard steel, the intermediate grade being medium steel. These grades are usually defined by their ultimate strength.

Process of Manufacture. Steel is made by the Bessemer or open hearth process. Open hearth steel is now used for all important bridge and building work. It is almost invariably required when a regular specification governs the work.

Bessemer steel is not so uniform in quality as open hearth steel and is, therefore, not so reliable a material. It is made in a

converter, by blowing a blast of air through molten pig iron until the carbon and silicon are all burnt out. Ferro-manganese is then added to recarbonize the metal the required amount and to absorb the excess of oxygen, which would make the steel "rotten." From the converter the metal is poured into a ladle and then into moulds. Here the metal is allowed to solidify, producing ingots. The ingots are reheated and then rolled into slabs, blooms and billets of various sizes, depending upon the final form into which they are to be rolled.

Open hearth (Siemens-Martin) steel is made in an open hearth furnace and is of two kinds, acid and basic. The use of the latter predominates.

Acid steel is made in a furnace having a lining of a refractory silicious material which has an acid reaction. Basic steel is made in a furnace having a lining of magnesite or dolomite, which has a basic reaction. The raw material in either case is pig iron, now usually charged in a molten state. If the pig iron contains larger percentages of phosphorus and sulphur than are allowed in the steel, these must be reduced. Since they have a great affinity for iron, some reagent must be introduced in the molten metal which has a greater affinity for them. For this purpose a basic material must be used, since they form acids when oxidized. The material employed is lime charged in the form of limestone. This cannot be used in an acid lined furnace because it would form a flux with the lining. In the acid process, therefore, the percentages of phosphorus and sulphur in the raw material must not much exceed what is allowed in the finished product. This process requires a better grade of pig iron than the basic process.

In the open hearth *basic process,* large quantities of oxide of iron in the form of iron ore or mill scale, are charged into the furnace, together with limestone and molten pig iron. The lime which is formed, combines with part of the phosphorus and sulphur, reducing the percentages of these to minute quantities, while the oxide of iron serves to burn out the carbon, manganese and silicon. The desired amounts of carbon and manganese are then added. As in the Bessemer process, the steel is made into ingots, billets and slabs, and is finally rolled into the desired shapes.

Upon the care with which all of these processes are carried out depends the uniformity and finish of the rolled steel, as well as its quality to some extent. Lack of uniformity may be due to unequal heat treatment, unequal working or to segregation in the ingot, producing a steel of variable composition. If a piece of steel is not uniformly treated and cooled there may be internal stresses in it, hence it is required that pieces which are heated in working as eye bars, for example, must be annealed. The hotter a piece of steel is heated, the coarser grained it will be and the more rapidly it is cooled, the harder it will be. Rolling and hammering steel increases its density and strength, but working it at a blue heat may crush the grain, which is very injurious. Rough iron cracks are indications of "red shortness" or burning. Other surface defects are easily discovered on close inspection. Defects due to bad heat treatment, improper working, excess of impurities, etc., are discovered by chemical analysis and physical tests. Tests are made of material from each heat, melt or blow.

Specifications allow a range of 8,000 to 10,000 lbs. per sq. in. in the ultimate tensile strength of any particular grade of steel, but they do not all agree as to the limits. Depending upon the specifications, soft steel may include steel having an ultimate strength as low as 52,000 lbs. per sq. in. and as high as 62,000 lbs. per sq. in., medium steel as low as 60,000 and as high as 70,000 lbs. per sq. in., and rivet steel from 48,000 to 58,000 lbs. per sq. in.

The ultimate strength, elastic limit, elongation and reduction of area are determined from test pieces cut from the finished material. It is usually required that the elastic limit must not be less than one half of the ultimate strength. As it is not difficult to obtain an elastic limit equal to 0.6 of the ultimate strength it would be well to specify, as is sometimes done, a minimum for the elastic limit. Thus, if the ultimate may vary from 60,000 to 68,000 lbs. per sq. in., the minimum elastic limit might be fixed at 32,000 lbs., for example, and then steel having a greater ultimate than 64,000 lbs., would have to have an elastic limit of at least one half the ultimate. In this case the unit stresses could be based upon an elastic limit of 32,000 lbs.

In commercial testing, the elastic limit is determined by the drop of the beam of the testing machine, that is, it is really the yield point.[1]

It is important that steel be ductile and not brittle. For determining ductility, the tension piece is measured after rupture to determine the stretch in an original length of eight inches, and the reduction of area at the point of fracture. The determination of the latter is not important and is usually omitted. In a general way, the elongation in eight inches is about 30% for good steel having an ultimate strength of 56,000 lbs. per sq. in., and 25% for 65,000 lb. steel. This is for standard test specimens having an area of cross section of not less than ½ sq. in. The usual requirements are 25% for soft steel and 22% for medium. Pin material is only required to have an elongation of 16% or 17%, and eye bars, when tested *full size*, 10% in the body of the bar.

The percentage of reduction of area is about 1.8 to 1.9 times that of the elongation.

The *appearance of the surface of the fracture* is always noted in testing steel. If it shows defects such as blisters, cinders, spots, cracks or lack of uniformity of color, it is not a desirable product. The fracture of good material is described as "silky," and has a "uniform, fine grained, structure of blue steel gray color, entirely free from fiery 'luster' or a 'blackish' cast." If the fracture is granular and has a 'fiery" lutser it indicates over heating. If the fracture is dull or "sandy," the steel is impure, or worked cold, and should be rejected.

The *bending test* requires that the test piece shall bend cold without sign of fracture. For soft steel it must bend flat on itself, and for medium steel, to a curve whose diameter is from one to three times the thickness of the piece tested. Some specifications require this bending test to be made upon pieces which have been heated and quenched in water.

A *punching test* is sometimes specified. This requires that the walls between the punched holes shall not break down except when they are less than ¼ inch thick.

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[1]See Heller's "Stresses in Structures," Art. 21.

The *drifting test* requires that the ductility of the metal must be such that a punched hole will stand drifting until its diameter is increased from 33⅓ to 100% without cracking the edges.

Pieces of large cross section are apt to be "piped," that is, they are not solid. This is due to bad working or unequal cooling, and occurs particularly in pins. It is usually specified that the larger sizes of pins (say over 4½ inches) must be forged from blooms having a sectional area of about three times the area of the pin, in order that the material may be sufficiently worked to make it sound.

Full sized tests are usually confined to eye bars. Some reductions in requirements of ultimate strength and elongation are made from those required for small test pieces, because pieces of large cross section do not test as high as those of small cross section.

33. Inspection. If an inspector is employed on a contract, his duties may relate to material, shop work and erection. On some classes of work the purchaser employs no inspector. The manufacturer has all work inspected as to dimensions, to avoid trouble in erection. In some cases reports of tests of material are furnished by the rolling mill, which conducts a testing department for this and other purposes.

An inspector is most frequently employed to make tests of the steel, at the mill, as it is rolled. The shop work is also inspected for the best classes of work, but in comparatively few cases is the field work inspected by a regular inspector. The large railway companies have inspectors in the mills, the shop, and the field.

The inspection of material includes tests, analyses, surface inspection, measuring sizes, etc., of steel, lumber, paint, etc. A shop inspector should see that no material is injuriously treated, that reaming of rivet holes is properly done, that all parts are made in accordance with the drawings, that rivets are good and tight, that members are straight, that no work is ragged or unfinished, that painting is done in accordance with the specifications, and should make reports of progress.

The field inspector should see that no part of the structure is injuriously treated, that no members are interchanged, that all field driven rivets are good, and that the painting is properly done.

Rare qualifications are required for a good inspector. He must serve his employer honestly and avoid friction with the contractor.

CHAPTER IV.

ROOFS.

The roofs of buildings in which it is not desirable to have columns at frequent intervals, are supported by means of trusses which are in turn carried either on columns at the sides or on masonry walls. These trusses may be either of steel or a combination of wood and steel. Steel trusses are usually used in steel or brick, mill and factory buildings, and in fire proof buildings. Combination trusses are used in wooden buildings requiring a large floor space free from columns, and often also in large brick and stone buildings, such as churches and public buildings of all kinds.

34. Construction. The roof trusses are placed transversely of the building, their distance apart (See Art. 37) depending on the length of span, type of construction of the building in general and the kind of roof covering. The upper inclined members of the trusses, parallel to the slope of the roof are called *rafters*. (See Fig. 31.) Longitudinal beams extending from truss to truss are supported by the trusses at intervals along the rafters. These are *purlins,* and they carry the roof covering, either directly or by means of boards, called *sheeting* or *sheathing,* running transversely to the purlins (up and down the roof) or diagonally across them.

35. Roof Coverings. For mill buildings, the commonest kinds of roofing are *corrugated steel or iron, slate, tile, tin* and various patent sheet metal roofs, *tar and gravel* and similar patented combinations.

The corrugated steel or iron is usually fastened directly to the purlins by means of clips.[1] *Slate* is usually nailed to sheeting boards with a layer of roofing felt between, although sometimes heavy slate is fastened directly to small purlins placed about 10½ inches apart. Tile is usually fastened directly to

[1]See "General Specifications for Steel Roofs and Buildings," by C. E. Fowler, Figures page 17.

angle purlins spaced about 13 inches apart, without any sheeting. *Tin, and similar types* of roofing are laid on sheeting with roofing felt between. *Tar and gravel* roofs are laid on wooder sheeting or sometimes on reinforced concrete slabs.

The main function of the roof is to shed water, and in order to do this without leakage, it must have a fall or slope. The amount of slope required depends upon the kind of roof covering.

The *pitch* of a roof is the ratio of its rise to its span. Thus for a 60 ft. span, if the rise is 15 ft., the pitch is ¼, if the rise is 20 ft. the pitch is ⅓. The *least pitch* advisable to use with *corrugated steel slate or tile* is about ¼, that is a fall of about 6 inches per foot, and this is the pitch used for most mill and factory buildings. *Tin* and similar roofs with *water tight joints* may have a fall of as little as ½ inch per foot. *Tar and gravel* roofs should have a fall of from ¾ in. to 2 in. per foot.

36. Types of Trusses. It is only in unusual structures such as train sheds, exposition buildings, grand stands, etc., that the span of a roof truss exceeds 100 ft. The slope of the roof and local conditions such as required clearances, ventilation, light, etc., will usually determine the general outline of the truss. Any type of bracing may then be selected to suit the materials of construction.[1]

For roofs of ordinary pitch and span, the Fink truss is by far the commonest type. Figure 31 (a), (b), (c) and (d) shows several modifications of this form of truss, to suit spans of varying length. For sake of economy in the rafters it is not desirable to have many loads coming on them from the roof purlins, between panel points, hence the advisability of increasing the number of panels as the span increases.

For combination trusses of wood and iron, one of the forms shown in Fig. 31 (g) and (h) is used. In these trusses the diagonal compression members and the top and bottom chords are made of wood, and the vertical ties are rods.

For *flat roofs* some form of truss must be used similar to Fig. 31 (f), (h) and (l), in order to gain sufficient depth at the center to give economic chord sections.

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[1]See Heller's "Stresses in Structures," Art. 117.
See Ketchum's "Steel Mill Buildings," page 146.

Another type of roof which is rapidly coming into favor is the "*saw-tooth*" roof, shown in Fig. 31 (k). The plane of the steeper rafter is glazed, and this side is made to face the North if possible. By this arrangement the floor below is lighted by an even diffused light, without the necessity of making the building narrow in order to gain light from the sides, and without the disadvantages of sky lights through which the direct rays of the sun may shine.

Roof trusses for very long spans are usually three-hinged arches, the lower hinges being connected by bars under the floor to take the thrust.[1]

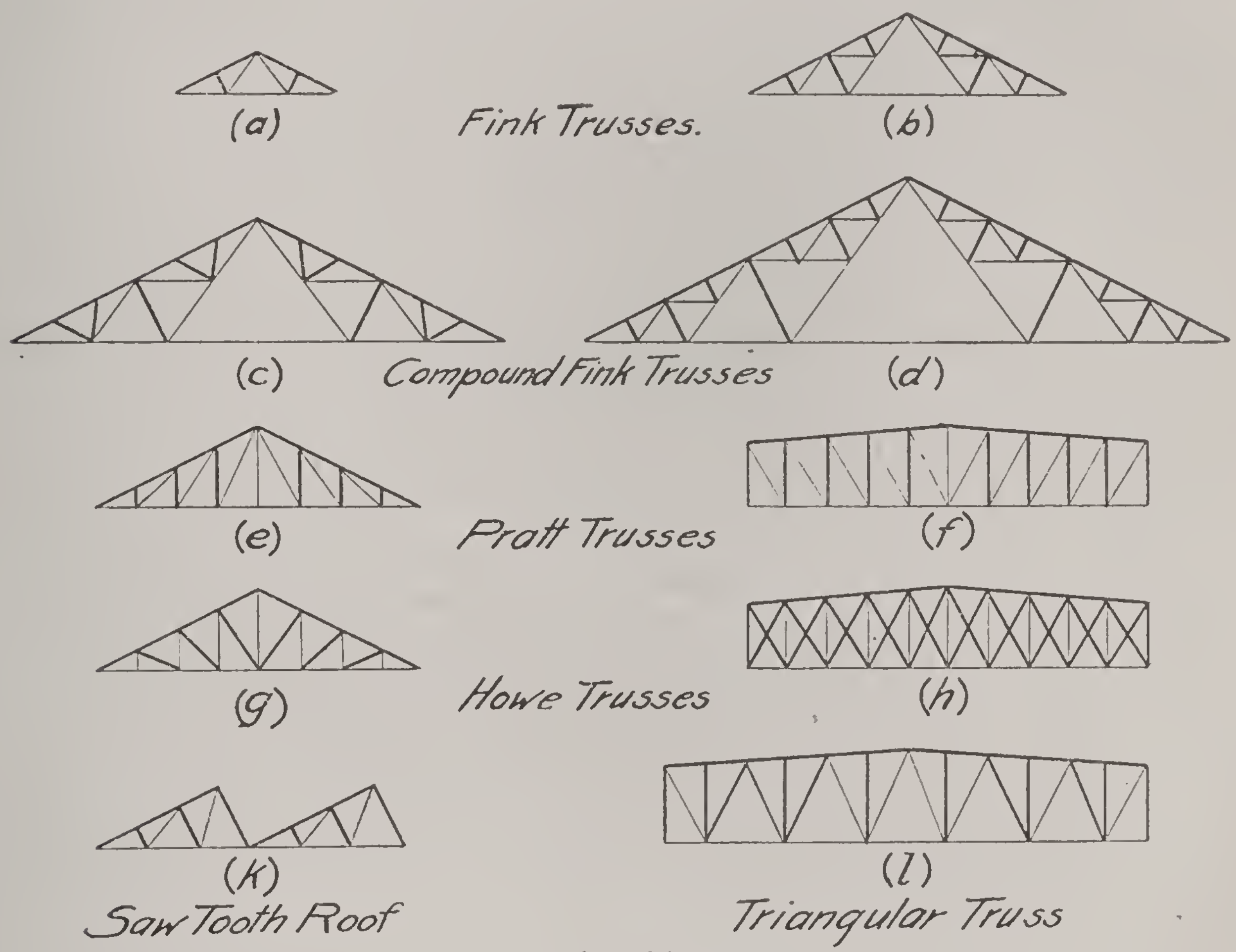

Fig. 31.

The roof trusses of grandstands usually project beyond their supports at both ends. These are called cantilever trusses.

Sometimes for the sake of appearance or to gain clearance, the lower chord of a roof truss is curved upward. This always increases the cost and weight very materially.

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[1]See trainsheds for Penna. R. R. at Jersey City and Philadelphia. and of the Phila. and Reading R. R. at Philadelphia, in Eng. News, Vol. 26, p. 276 and Vol. 29, pp. 507 and 508, Vol. 42, p. 212.

Ordinary roof trusses are made with riveted connections, because such construction is cheaper and gives greater rigidity than the pin connection. Heavy trusses of long span are sometimes made with pin connections, because the saving in cost of erection is more than the saving in shop work with riveted connections. The members of a pin connected truss offer a smaller percentage of area to the corrosive action of gases than those of riveted trusses. Provision is sometimes made for the weakening effect of corrosion, by increasing the thickness of material above that required to take the actual stresses or by adding a certain percentage to the loads.[1]

In calculating the relative economy of roofs of different pitches, the roof covering must be taken into account, as the greater the pitch, the greater the area of roof covering. Corrugated steel usually makes the cheapest roof, as the dead weight is small. The most expensive roofs are those of tile and heavy slate, laid directly on the purlins.

37. Building Construction. Trusses carrying light roofs are usually spaced from 16 ft. to 20 ft. center to center. Theoretically the shorter this spacing, the less the total weight of trusses and purlins, per sq. ft. of covered area, but on account of practical limitations in the size of materials, etc.,[2] and on account of the greater cost per pound for the manufacture of trusses, than for purlins, the spacing of the heaviest trusses is very seldom less than 10 ft. center to center. When the weight of the roof covering is very great, the purlins are sometimes supported between trusses, on beams called *"jack rafters,"* which are supported at the ridge and eave, on longitudinal beams, carried by the trusses.

For a building with masonry walls, no wind bracing is necessary unless the end walls do not run up to the roof, but some bracing is usually put in to facilitate erection and to stiffen the roof.

In steel buildings bracing is necessary to provide for both longitudinal and transverse wind forces, and to stiffen the building in case there is any vibration due to live loads, shafting or machinery.

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[1] See Fowler's Specifications for Steel Roofs and Buildings, Art. 10.
[2] See Fowler's Spec. Arts. 37, 39, 44, 45, 59, and 64.

With the ordinary Fink truss, the transverse bracing consists of knee braces connecting the trusses and columns, as shown in Fig. 32, the wind load being carried to the foundations by bending in the columns. If a truss is used having some depth at the ends, (see Fig. 31 (f), (h), (l),) the knee braces may be dispensed with and the columns run through to the rafter.[1]

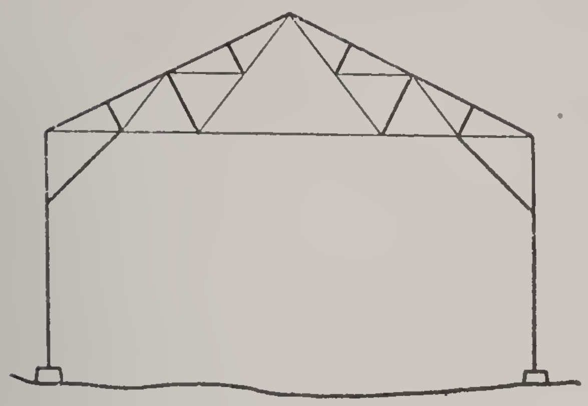

Fig. 32.

Longitudinal bracing may be put in, in three planes. That in the plane of the rafters is called *rafter bracing*, that in the plane of the bottom chords is called *bottom chord bracing*, and that in the vertical planes between the columns is called *side bracing*.

Theoretically, only one panel of longitudinal bracing is necessary to take care of the longitudinal wind forces, but for convenience in erecting the steel work, not less than two panels are braced, and in long buildings the braced panels are not farther apart than three or four panels. This arrangement usually requires less material in the bottom chord and rafter bracing diagonals than is given by the smallest size of rod ever used, and these members are therefore usually made of ¾ inch round rods. Struts are required between the trusses and columns as members of the lateral systems. In the rafter bracing, the roof purlins are usually made to serve the purpose of struts. Several lines of ties are put between the bottom chords of the trusses in the unbraced panels. These serve to reduce the vibration of the roof, especially when cranes or hoists are attached to the trusses. The general arrangement of the bracing is shown in the stress sheet, Fig. 35.

38. Loads. A roof truss ordinarily carries nothing but dead loads, which includes wind and snow loads and the weight of the structure itself, such as the covering, sheeting, purlins, trusses, bracing, ceiling, shafting, etc. A traveling hoist carried by a roof truss would constitute a live load.

[1]For figuring stresses in Columns see Heller's "Stresses in Structures," Chap. X and XIII.

The pressure of a gas or liquid is always normal to the surface on which it acts, consequently the wind load acts normally to the surface of the roof. All other loads act vertically, and are estimated in pounds per horizontal square foot. The wind is assumed to blow *horizontally,* the pressure which it exerts depending upon its velocity. An empirical formula frequently used is $W=0.004V^2$ in which W is pressure per sq. ft. in pounds, on a surface perpendicular to the direction of the wind, and V is velocity in miles per hour. Experiments extending over seven years at the site of the Forth bridge in Scotland. show that the pressure on large surfaces is much less per sq. ft. than on small ones. The maximum pressure recorded on a surface of 1½ sq. ft., was 41 lbs. per sq. ft., and on a surface of 300 sq. ft., was only 27 lbs. per sq. ft.

The roof of a building presents a large surface, and is usually figured for a wind load due to a *horizontal* force of 30 lbs. per sq. ft. The sides and ends of buildings are usually figured for a maximum wind pressure of from 20 to 30 lbs. per sq. ft., but this pressure is often taken as low as 10 lbs. per sq. ft.[1]

The normal wind pressures on roofs of various pitches for a horizontal wind force of 30 lbs. per sq. ft. are given in Fowler's Specifications for Steel Roofs and Buildings, Art. 6. These are based on the empirical formula,

$$W' = W \sin a^{(1.84 \cos a - 1)}$$ in which

W' = normal pressure per sq. ft.

W = horizontal pressure per sq. ft.

a = angle of inclination of the roof with the horizontal.

In the article referred to, the columns marked "*Vertical*" and "*Horizontal*" are not the ordinary components of the normal given, but they do not differ much from them.

When a roof truss rests on masonry walls, one end must be free to move longitudinally, in order to provide for changes in length due to temperature changes. This is usually arranged by providing slotted holes in one end for the anchor bolts, thus allowing the truss to slide on the bed plate. For long trusses rollers are provided to reduce the friction where this movement

[1]See Fowler's Specifications, Art. 7.

takes place. If we assume that there is no friction at the expansion bearing, the reaction must be vertical at that point and therefore it is necessary to calculate the stresses in the truss with the wind blowing from both directions. When trusses are fastened rigidly to the top of columns, the horizontal components of the wind reactions are sometimes assumed to be equal and sometimes the wind reactions are assumed parallel. In the latter case it is only necessary to calculate the stresses for the wind blowing in one direction. *A vertical equivalent wind load* is sometimes used together with the other loads, as explained below.

The *snow load* varies with the climate, the slope of the roof and the roughness of the roof covering.[1] The weight of freshly fallen snow is from 5 to 12 pounds per cu. ft.[2] The snow load may act on one side only of a roof, as a heavy wind or the sun on the other side would dislodge it. When the pitch of the roof is variable, as it frequently is for train sheds, snow might stand on only a part of either or both sides, and might be a variable load. It is not usually assumed that the maximum wind and snow loads can act upon one side of a roof at the same time, because the wind would dislodge the snow.

We may therefore have a partial snow load, a partial wind load or a combination of these, in addition to the weight of the structure itself.

For the ordinary Fink roof truss of 1/3 pitch or less, none of these partial loads give maximum stresses, and an equivalent wind and snow load is usually taken as acting vertically over the entire roof. This simplifies the calculation of stresses, and is used whether or not the trusses rest on rollers at one end.[3]

The weight of the roof covering should be calculated, remembering that the weight per horizontal square foot is equal to the weight per sq. ft. of the roof surface multiplied by the secant of the angle of inclination of the roof with the horizontal.[4]

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[1]See Fowler's Specifications, Art. 5.

[2]See Trautwine's "Civil Engineer's Pocket Book," p. 384.

[3]See Fowler's Specifications, Art. 12.

[4]For weights of various roofing materials, see Trautwine's "Civil Engineer's Pocket Book." See also Fowler's Specifications. Art. 8.

The thickness of the sheeting when used depends upon the spacing of the purlins. It varies from $\frac{7}{8}$ in. to 2 in. in thickness, and may be calculated, using an extreme fiber stress of from 1200 to 1500 lbs. per sq. in.[1]

The weight of the purlins may be calculated after they are designed, which is usually done before the trusses are figured. For corrugated iron roofs they will usually amount to about 3 lbs. per horizontal square foot.

The weight of the trusses may be estimated from a comparison with a similar building which has been designed, or it may be approximately obtained from an empirical formula.[2] After the design is completed, an estimate of the weight is made and the dead load used in the calculations is verified. If this differs materially from the amount used, corrections in the design should be made.

39. Stresses. The stresses in any statically determinate structure[3] may be calculated from the principles of statics.[4] For ordinary trusses the loads and reactions are all taken vertical. Since trusses of the same type with the same number of panels are similar figures, the stresses in them are proportional, for different spans, to the panel loads. Tables of stresses in various types of trusses for panel loads of one pound are given in various hand books.[5] The stress in any member of a truss similar to any of these is gotten by multiplying the coefficient given, by the panel load. This is readily done on the slide rule.

40. The Design of a Roof. To illustrate the method of procedure we will now give a complete design of a roof. The following data will be assumed:

The extreme width out to out of pilasters will be 81 ft. 1 in.

The extreme length of building will be 221 ft. 1 in. This will give a length of 220 ft. center to center of end walls, assuming the walls to be 13 in. thick, and a width of 80 ft. center to center.

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[1] See Fowler's Specifictions, Art. 22.

[2] See Fowler's Specifications, Art. 9.

[3] See Heller's "Stresses in Structures," Art. 42.

[4] See Heller's "Stresses in Structures," Chapters III, IV, V and VI.

[5] See Fowler's Specifications, pages 10 to 15. See also Carnegie's Pocketbook, page 174.

The end walls will run up to the roof and carry the end panel purlins.

We will use 11 bays at 20 ft.=220 ft.

Roof covering to be No. 20 Corrugated Steel.

Specifications to be Fowler's "*Specifications for Steel Roofs and Buildings,*" 1904 edition.

Pitch of the roof to be one fourth.

The *stress sheet,* Fig. 35, gives a general outline of the arrangement of the purlins, bracing, etc.

The student should familiarize himself with the specifications and refer to them constantly.

Loads:—Snow (Spec. Art. 5)	15 lbs. per horiz. sq. ft.
Wind (Spec. Art. 6) Vertical	15 lbs. per horiz. sq. ft.
Corr. Steel No. 20 (Spec. Art. 8)	2 lbs. per horiz. sq. ft.
Purlins say	3 lbs. per horiz. sq. ft.
Total carried by the Purlins	35 lbs. per horiz. sq. ft.

For corrugated steel No. 20 the roof purlins must not be spaced over 4 ft. 6 in. center to center (Spec. Art. 27). The extreme length of the rafter is $\sqrt{(40.5)^2+(20.25)^2}=45.3$ ft. If we use 11 purlins, their distance center to center will be almost exactly 4 ft. 6 in. This arrangement will not make the purlins come at the panel points of the truss (See Fig. 35) but this cannot be avoided, hence the rafters must also act as beams to carry the purlin loads to the panel points of the truss, as well as members taking the regular truss stress.

The number of horizontal square feet tributary to each purlin is $\frac{81}{20}\times 20=81$ sq. ft., which at 35 lbs. per sq. ft. gives 2835 lbs. total load on each purlin. The maximum moment $=\frac{WL}{8}=\frac{2835\times 20}{8}=7{,}088$ ft. lbs.$=85{,}050$ in. lbs. The maximum allowed extreme fiber stress for purlins is 15000 lbs. per sq. in. (Spec. Art. 19).

$$\frac{M}{s}=\frac{I}{v}=\frac{85050}{15000}=5.67=\text{required section modulus.}$$

The lightest I beam with a section modulus greater than 5.67, is a 6 in. I x 12¼ lbs. (See Cambria, page 160). The lightest channel having the required section modulus in a 7 in. chan-

nel $9\frac{3}{4}$ lbs., the lightest angle that can be used is a 7 in. x $3\frac{1}{2}$ in. x $\frac{1}{2}$ in., which weighs 17.0 lbs. per ft. The I beam would be better than the channel, because it is considerably stiffer sidewise, but it weighs considerably more. Channels are usually used in such positions and we will use the channel.

The above method of calculating purlins is not correct, since the moment of inertia which we used in the calculation is not about an axis perpendicular to the plane of the loads.[1] It is, however, close enough if the purlins are held from deflecting in the plane of the roof, and is the method always used. To prevent the purlins from sagging and to take the component of the load parallel to the roof, sag ties are inserted at distances not more than about 30 times the width of the purlin apart. (Spec. Art. 42.) These are usually made of $\frac{5}{8}$ in. round rods threaded at the ends, which are run through holes in the purlin webs with nuts to hold them in place. They are carried across the ridge in such a manner that the loads on the two sides of the roof balance each other.

The purlins are fastened to the rafters by means of angle clips as shown in the truss drawing, Fig. 36 (Spec. Art. 49). The clip should be below the purlin to facilitate erection.

We can now make an estimate of the weight of our purlins.

One purlin weighs $9\frac{3}{4}\times 20=195$ lbs. 22 purlins$=22\times 195=4290$ lbs. per bay. Sag ties$=3\times 2\times 46=276$ lin. ft. $276\times 1.04=287$ lbs. per bay. About 10% should be added to this for nuts and laps, making 320 lbs. per bay. Total weight then is $320+4290=4610$ lbs. per bay. This is distributed over $81\times 20=1620$ sq. ft. The weight per sq. ft. then is $\frac{4610}{1620}=2.85$ lbs. per sq. ft. Our estimate was 3 lbs. per sq. ft.

The weight of the trusses may now be calculated approximately from the formula given in Art. 9 of the specifications. $0.04\times 80+0.4=3.6$ lbs. per sq. ft. Calling this 4 lbs. we have $35+4=39$ lbs. per horiz. sq. ft. for the truss load. For the purlin spacing which we have, no two panel loads on the truss will, in general, be the same, but it is sufficiently close to assume

[1]For a complete discussion of this subject see Heller's "Stresses in Structures," Art. 69.

them all equal for the stresses in the truss. The panel load then will be 10×20×39=7800 lbs.

By means of the table on page 13 of the specifications, we find the following stresses for a panel load of 7800 lbs. (Note that the lettering of the truss in the specifications is not the same as used here.)

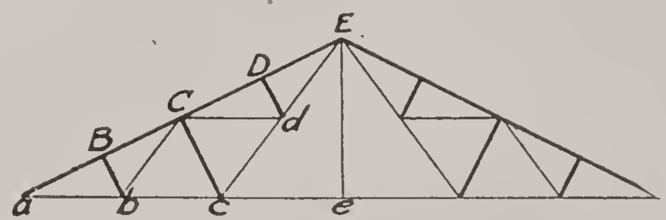


Fig. 33.

Mem.	Stress	Allowed Unit	Req. Area	Material used	Rad. Gyr.	Length	Actual Area
aB	+61,100	15000		Bending and Direct Stress combined 2 Ls 6"×3½"×½"		11.3	
BC	+57,600					11.3	
CD	+54,100					11.3	
DE	+50,600					11.3	
ab	−54,600	15000	3.64	2 Ls 3½"×3½"×5/16" 5/8" Riv.			3.71 Net
bc	−46,800	"	3.12	" " " " " " "			" "
ce	−31,200	"	2.08	2 Ls 3"×2½"×¼" " " "			2.27 "
Bb	+7,000	7,900	0.89	2 Ls 2"×2"×¼"	0.61	5.6	1.88
Cc	+14,000	6,550	2.14	2 Ls 3"×2½"×¼"	0.95	11.3	2.64
Dd	+7,000	7,900	0.89	2 Ls 2"×2"×¼"	0.61	5.6	1.88
bC	−7,800	15,000	0.52	2 Ls 2"×2"×¼" 5/8" Riv.			1.50 Net
Cd	−7,800	"	0.52	" " " " " " "			" "
cd	−15,600	"	1.04	2 Ls 2½"×2"×¼" " " "			1.76 "
dE	−23,400	"	1.56	" " " " " "			" "
Ee	0	—	—	2 Ls 2"×2"×¼"			

The tension members may be proportioned first. The smallest angle allowed is 2 in. × 2 in. × ¼ in., (Spec. Art. 64), and all members should be symmetrical (Spec. Art. 39 and 40). Therefore the smallest member allowed will be 2 *Ls* 2″×2″×¼″. The largest rivets allowed in these angles are ⅝ in. (See Cambria, page 54). The gross area is 2×0.94=1.88 sq. in. The net area is 1.88−(2×¼×(⅝+⅛)=1.88−0.38=1.50 sq. in. This will answer for *bC*, *Cd* and *cd*. but *cd* and *dE* are usually made continuous and should therefore be the some size. 2*Ls* 2½″×2″×¼″ will answer for these, the net area being 2×1.07−2×¼×(⅝+⅛)=1.76 sq. in. For light trusses *ab* and *bc* are also usually made continuous. The sizes of the other tension members are easily determined, as shown in the table above.

We will try to make each of the compression members of two angles. These will be back to back, and will be far enough apart to admit the connection or gusset plates between them at the joints. We will try to make all gusset plates $\frac{3}{8}$ in. thick. The radius of gyration for the various sizes of angles may be taken from *Cambria,* pages 189 to 193. The least width of compression member allowed is $\frac{1}{50}$ of the length (Spec. Art. 59), therefore $2'' \times 2'' Ls$ cannot be used in compression members whose length is greater than 100 in.$=$8.3 ft. For *Bb* and *Dd* we will try $2Ls 2'' \times 2'' \times \frac{1}{4}''$. The least radius of gyration is 0.61. The maximum allowed units tress is $12500 - 500 \cdot \frac{5.6}{0.61} = 7900$ lbs. per sq. in. The required area will be $\frac{7000}{7900} = 0.89$ sq. in., while the actual area is 1.88 sq. in., and therefore *Bb* and *Dd* may be made of $2Ls\ 2'' \times 2'' \times \frac{1}{4}''$.

For *Cc* the least allowable width of member will be $\frac{11.3 \times 12}{50} = 2.71$ in. The least angles that can be used will be $2Ls\ 3'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ unless "special" angles are used, which usually take longer for delivery from the mills and might thus delay the work. The table of column unit stresses on page 15 of the specifications may be used instead of applying the column formula each time. Trying $2Ls\ 3'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ for *Cc*, the least radius of gyration$=$0.95 and $\frac{l}{r} = \frac{11.3}{0.95} = 11.9$. Allowed unit stress from table$=$6,550 lbs. per sq. in. The required area$=$ $\frac{14000}{6550} = 2.14$ sq. in. The actual area$=2 \times 1.32 = 2.64$ sq. in., which is sufficient.

The rafter should be made continuous from eave to ridge, if this length is not too great, say over 60 ft. It must be proportioned for direct compression and bending.[1] The maximum compression occurs in *aB,* and the bending in this case is also a maximum in this panel, or nearly so. *aB* is loaded transversely by the purlins, as shown in Fig. 34. Considering the

[1]For a complete discussion of this subject see Heller's "Stresses in Structures," Art. 111.

member as a simple beam supported at a and B, the maximum moment will be $2509 \times 3.55 = 8907$ ft. lbs.

The rafter is not really in the condition of a beam simply supported at the ends, nor are the ends fixed, because the connections are elastic. The actual moment lies somewhere between that for the two conditions of free and fixed ends, (Spec. Art. 16) and may safely be taken as $\frac{5}{8}$ of the moment for a simple beam. We have then $M = \frac{5}{8} \times 8907 =$ 5567 ft. lbs. = 66800 in. lbs.

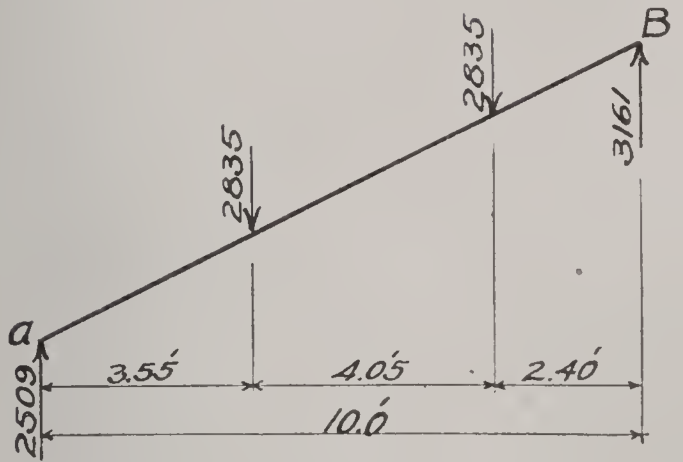


Fig. 34.

This is the positive moment under the load nearest the middle. There is also a negative moment at each support which may be assumed to be equal to the same amount, consequently in applying the formula of Art. 16 of the Spec., the factor n must be taken as the *greatest* distance from the neutral axis to the extreme fiber.

$s = \frac{Mn}{I} + \frac{P}{A}$ We will try $2Ls\ 6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$. $I = 2 \times 16.59 = 33.18$. $n = 3.92$. $A = 2 \times 4.5 = 9.0$. $s = \frac{66800 \times 3.92}{33.18} + \frac{61100}{9.0} = 7891 + 6789 = 14680$ lbs. per sq. in. The allowed fiber stress is 15000 lbs. per sq. in., therefore these angles are large enough. If the next size smaller angle be tried it will be found too small.

The laterial struts do not carry much stress, and their size is determined by Art. 59 of the specifications, which requires that their least width be not less than $\frac{1}{50}$ of their length. This requires the use of members not less than 4.8 in. wide. The most economical section will be $2Ls\ 5'' \times 3'' \times \frac{5}{16}''$, the longer legs being back to back. The ridge strut is usually made the same for all bays. The bottom chord ties take no definite stress, but should not be less than one angle $3\frac{1}{2}'' \times 3'' \times \frac{5}{16}''$ with the $3\frac{1}{2}$ inch leg vertical. All the laterals may be $\frac{3}{4}$ in. round rods. The vertical member of the truss at the middle takes no stress, but keeps the lower chord from sagging.

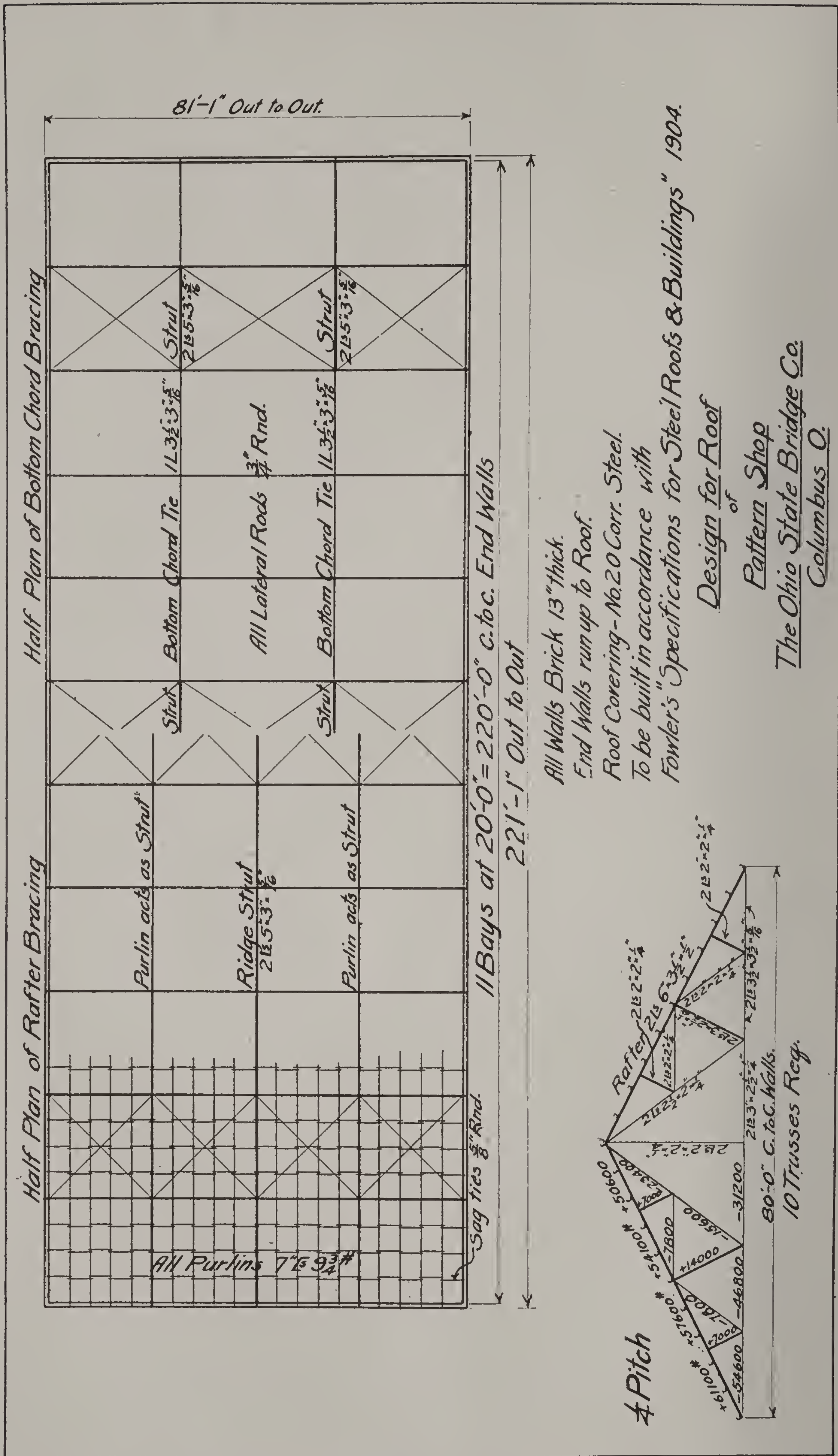

81'-1" Out to Out.
Half Plan of Rafter Bracing
Half Plan of Bottom Chord Bracing
Purlin acts as Strut
Ridge Strut
Bottom Chord Tie
All Lateral Rods 3/4" Rnd.
Strut
All Purlins
Sag ties 5/8" Rnd.
11 Bays at 20'-0" = 220'-0" c.to c. End Walls
221'-1" Out to Out
All Walls Brick 13" thick.
End Walls run up to Roof.
Roof Covering - No. 20 Corr. Steel.
To be built in accordance with
Fowler's "Specifications for Steel Roofs & Buildings" 1904.
Design for Roof
of
Pattern Shop
The Ohio State Bridge Co.
Columbus O.
1/4 Pitch
Rafter
80'-0" C. to C. Walls.
10 Trusses Req.

Fig. 35.

41. The Detail Drawings. (27) Having completed the designing of the members, what is known as the *stress sheet* (21) is usually next made. This is ordinarily a line diagram as shown in Fig. 35, on which are written the stresses and sizes of all the members, as well as the general dimensions. After this is completed the shop drawings may be commenced.

To avoid eccentric stresses the center of gravity lines of all members coming together at a joint should intersect in one point. This is not practical in roof trusses because the drawings and templet work would be too complicated. Instead of using the gravity lines as *center lines,* the rivet gage lines are used. For members composed of $2Ls$ $2'' \times 2'' \times 1/4''$ the gravity line is 0.59 in. from the backs of the angles, while the rivet line is $1\frac{1}{8}$ in. out. This makes the eccentricity of the connection 0.535 in. For a member composed of two angles $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{5}{16}''$ the eccentricity is $2.00-0.99=1.01$ in. For an angle having two gage lines, the center should of course be taken on that rivet line which is nearest the gravity line. For a member composed of $2Ls$ $6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ the center of gravity is 2.08 in. from the back of the shorter leg. The rivet lines in a 6 inch leg may be spaced 2 in. and $4\frac{1}{2}$ in. from the back of the angle, and of course the inside gage line should be used as the center line.

In a roof truss it is the practice to have the *center lines* intersect in a single point at each joint except the shoe, joint *a*, Fig. 36. In order to take the reaction, the truss must have an appreciable depth at the end, consequently the center lines of the rafter and bottom chord are made to intersect at some little distance beyond the center of the bearing plate, as shown in Fig. 36. To facilitate the driving of the rivets, the depth at the end should be at least 6 inches. The distance from the center of bearing to the intersection is usually made such an amount as will avoid odd fractions of an inch in the center lengths.

Having determined this point by scale, the slope of the center line of the rafter is made *exactly* 6 in. vertical to 12 in. horizontal, for a $1/4$ pitch roof.

The usual standard size of shop drawings is $24'' \times 36''$. The border line should be a single heavy line placed $1/4$ inch from the line on which the tracing is trimmed. (Or see Fig. 25.)

The truss outline and the details are not drawn to the same scale. This amounts to drawing the detail of each joint separately and then assembling the joints into the form of a truss. It is not necessary to show the members broken between joints, although the distances are not to scale. Any rivet spacing between panel points cannot be drawn to scale, but it is not necessary to show all the rivets if the figures locating them are properly given. The scale for details is usually 1 in. or 1½ in. per ft., depending upon the available room. The larger scale is easier to work with, especially for the beginner. The scale for the center lines should be so selected that there will be sufficient room for the elevation of half the truss, the dimension lines, the top view of the rafter and the sectional view of the bottom chord, with all connections for purlins, laterals, etc. The scale for the details must be decided upon first. A sample drawing should be consulted and studied, and if necessary preliminary sketches made of the lateral connections. Compactness is desirable, but crowding, especially of dimension lines, should be avoided. Usually the scale for the outline should not be less than one half that of the details.

Usually on the same sheet with the truss there is put a diagram of the roof showing the location of trusses, bracing, etc., similar to Fig. 35. This is called an erection diagram. In a building with steel columns and other steel work, the erection diagram usually occupies a sheet by itself. Sometimes there is also sufficient room on the sheet with the truss drawing for drawings of struts, purlins, etc.

After the scales have been determined, one half the distance between the intersctions of the center lines of the rafters and bottom chord (40′—10½″ in our case) is laid off horizontally to the smaller scale chosen for the outline, and one half of this distance, if the roof has ¼ pitch, (20′—5¼″) is laid off vertically at the right end, this being the rise of the center line of the rafter above the bottom chord. Now the hypotenuse of the right triangle is drawn, which is the center line of the rafter. This center line is then divided into four equal spaces, and the center lines of the members *Bb*, *Cc* and *Dd* are drawn perpendicular to it. The intersections of *Bb* and *Cc* with the bottom chord determine points *b* and *c*, and of *Dd* with *cE*, point

d. The length of Cc is twice that of Bb and Dd, and equals aB. The lengths of ab, bc, bC, Cd, cd and dE are equal. Thus it is seen that the center lengths are very easily determined, as given in Fig. 36, from right triangles.

Joint a. Fig. 36 is a shop drawing of the truss. The forces acting at joint a are the rafter stress, the bottom chord stress, the purlin load and the reaction. There must be enough rivets in the joint to safely transmit these forces. To avoid changing punches, (6) the rivets will be made ⅝ in. throughout. (Maximum size for a 2 in. leg.) Joint a has larger stresses than any other, and if ⅜ in. connection plates were used throughout it would result in a large gusset here, consequently, (5) we will use a ½ in. gusset plate at a and make all the others ⅜ in.

The rafter stress is 61,100 lbs. The value of a ⅝ in. rivet in double shear is 6136 lbs. (This value is less than the bearing value on a ½ in. plate). The number of rivets required in the rafter connection$=\frac{61100}{6136}=10$ rivets. We will have to add an extra rivet to transfer the purlin load, this makes 11 rivets. The rivets immediately over the bearing plate in the bottom chord must transfer the vertical component of the rafter stress and the purlin load to the bottom chord angles which rest on the bearing plate.

The amount of this reaction will be $4\times7800=31200$ lbs., and the number of rivets required $=\frac{31200}{6136}=5$. In addition to these we must have in the bottom chord sufficient rivets to transfer the bottom chord stress, 54600 lbs. This requires$=\frac{54600}{6136}=9$ rivets, making a total of 14 rivets in the bottom chord connection. If the *reaction* acted equally on all the bottom chord connection rivets at this point, the connection would only have to be proportioned for the resultant of the two stresses $\sqrt{54600^2+31200^2}=62900$ lbs., and the rivets required would only be $=\frac{62900}{6136}=11$ rivets.

It is not good practice to put the rivets closer together than 2½ in., and they must not come nearer the edge of any piece than 1¼ in. (7). Care must be exercised to see that the rivets

in opposite legs of angles stagger so that one rivet head does not interfere with the driving of the other rivet.

Joint C. At this point the components of the stresses in bC and Cd, parallel to the rafter, balance each other in the plate and require no rivets in the rafter. The components perpendicular to the rafter must be transmitted to Cc. Their sum is 7000 lbs., which may be gotten by laying off the stresses and scaling the components parallel to Cc. The balance of the stress in Cc (=7000 lbs.) comes directly from the rafter. These together require $\frac{14000}{4690}$ =3 rivets in bearing on the ⅜ in. gusset plate. There must be a sufficient number of rivets through the rafter angles to transmit the 7000 lbs. More are put in here so as not to exceed the maximum allowed pitch.

It will be noted that the rivet spacing dimension line for each member starts at the *center.* A connection by a single rivet should never be used, and preferably at least three rivets should be used in any connection.

Joint c. A splice is made here in the bottom chord because it changes section, and it is made a field splice because the truss is too large to ship in one piece. Each truss is shipped in four pieces. The middle section of bottom chord and the vertical member Ee are two of these. The greatest depth of any piece will then be over 12 ft. at Cc, and this can not be handled by all railroads. (31)

The horizontal components of Cc and cd act toward the right and their sum, equal to 15600 lbs., must be transmitted to bc. (15600 lbs. is the difference in the stresses in bc and ce.) This requires $\frac{15600}{4688} = 4$ rivets in bearing on the ⅜ in. gusset plate. The bottom chord splice will require $\frac{31200}{3068} = 11$ rivets in single shear. We must have 11 rivets on each side of the splice, in the splice plate, which also acts as a connection for the bottom chord rods, strut and tie. The three rivets through ce and the gusset plate are not counted as a part of the splice (Spec. Art. 38) but are put in because there should be a connection between the vertical as well as the horizontal legs of the angles. The splice plate should have as much *net section* as the

angles of *cc*. Even with the minimum thickness of plate allowed there is a large excess in this case.

Lateral Connections. The lateral connections should be sufficient to take the full value of the area of a ¾ in. rod at 18000 lbs. per sq. in. (Spec. Art. 13). We have then $0.44 \times 18000 = 7900$ lbs. If we use a 1¾ in. pin for the lateral rods which have forked loops, the allowed bearing pressure of the pin on the ¼ in. plate will be $\frac{1}{4} \times 1\frac{3}{4} \times 25000 = 10900$ lbs.

It is very essential that clearance be provided where two members come together, except where tight joints are required, as in the bottom chord splice and at the ridge. The usual minimum clearance of members is ¼ in.

Rivet holes are made $\frac{1}{16}$ in. larger in diameter than the size of the rivet to be used. In roof work for lateral connections, pin holes are usually punched, and are made $\frac{1}{16}$ in. larger than the pin. Sizes of all rivets and holes must be plainly marked on the drawings.

The exact length of each piece must be given with its other dimensions. The length should invariably be given last. The width of a plate should be given first and the longer leg of an angle should be given first, thus, $1-15'' \times \frac{3}{8}'' \times 1'-7\frac{1}{4}''$, $2Ls\ 4'' \times 3'' \times \frac{5}{16}'' \times 21'-7\frac{1}{4}''$. The width of a plate should always be given in inches, and should not contain a fraction less than ¼ in.

For the use of the templet maker, the bevel of each inclined line of rivet holes should be given. The longer dimension of the bevel is usually made $1'-0''$. All dimensions, other than the widths of plates, of one foot or more should be given in feet and inches, and not in inches alone, thus $1'-1\frac{3}{16}''$ and not $13\frac{3}{16}''$.

While no dimension is ever to be taken by scale, off a shop drawing, it is nevertheless essential to *draw by scale*. This can not be done unless the drawing is worked up in a logical manner. A draftsman who makes many erasures will seldom become interested enough in his work to be a success. *Rivet heads and rivet holes should be drawn to scale.*

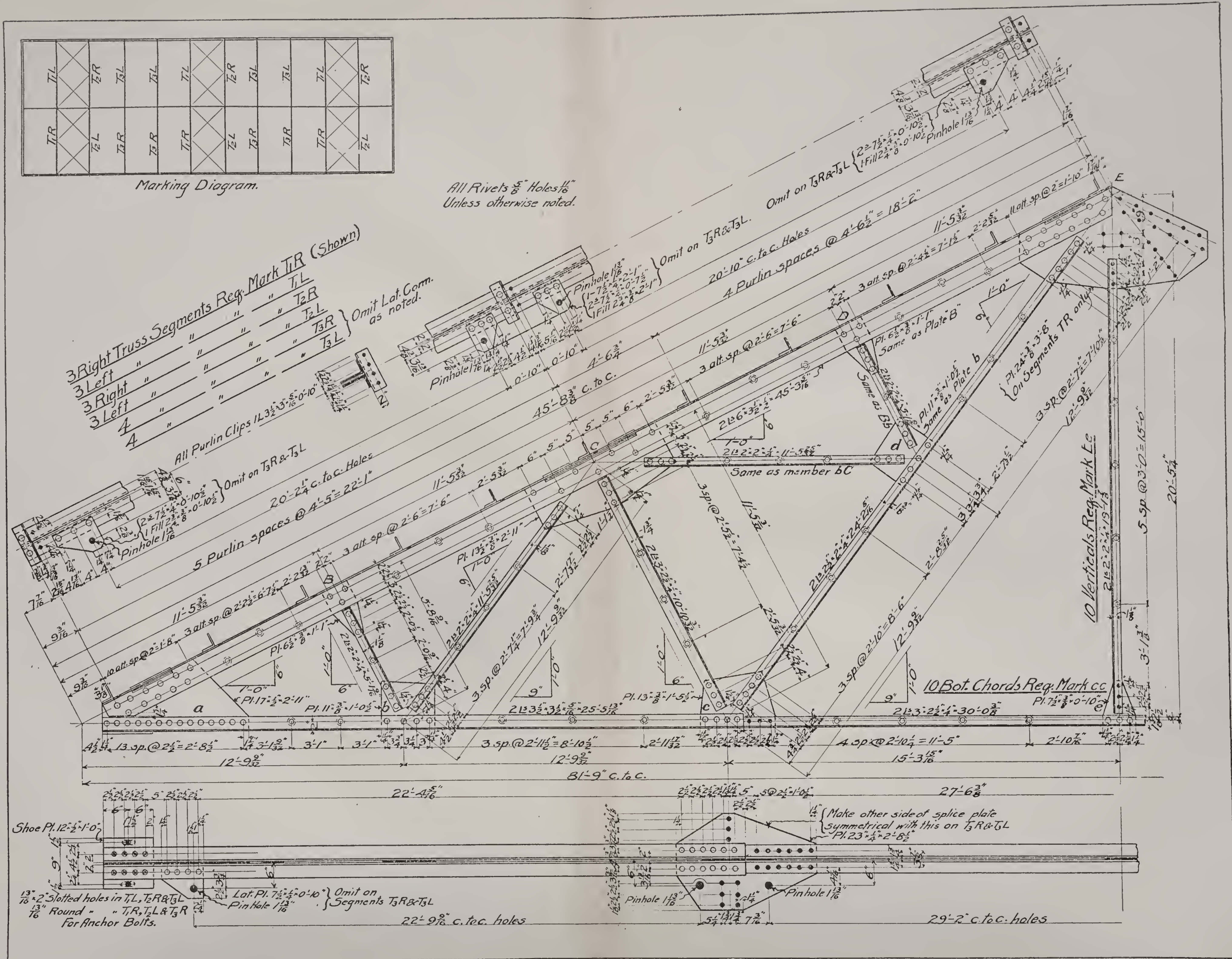
Accuracy is essential, but no smaller fraction than $\frac{1}{32}$ of an inch is ever used in structural work. If a line of spacing should add up $10'-6\frac{15}{32}''$, $10'-6\frac{1}{2}''$ *will not answer.*

All spacing must be continuous from center to center, and the different sets of spacing should be kept separate and in straight lines, if possible, not offset lines. First we have the general dimensions such as span and length of rafter, center to center; second, we have the distance center to center for each member; third, we have the rivet spacing which must be connected with the centers; and fourth, we have the open holes, which should be connected up for the benefit of the inspector. If there are holes in both legs of an angle, there must be two lines of spacing.

Each rivet hole must be located definitely, cuts on plates shown, and bevels of center lines given. The gages of all rivet lines must be given.

The good appearance of a drawing goes far to inspire confidence in its accuracy. It should be workmanlike. The appearance of a drawing depends largely upon the lettering and general arrangement. Except in the title, the letters should all be free hand and of a plain style. The figures should be particularly clear. Figures and letters should not all be the same size. The dimensions of a main member should be in larger figures than those for a detail, and center distances larger than rivet spacing. It is essential that the sizes of plates, angles, etc., be in the best possible place for them. Shop men are not supposed to be able to read a drawing as readily as a draftsman or templet maker. To become a proficient letterer, persistent practice is necessary, and will work a wonderful improvement in any man's work. (27)

The title and sheet number should be in the lower right hand corner, if possible. The name of the draftsman and the date when the drawing was finished, should appear in the title, as well as a statement of what is shown on the drawing. (See Fig. 24.)



Marking Diagram.
All Rivets 5/8" Holes 11/16"
Unless otherwise noted.
3 Right Truss Segments Req. Mark T1R (Shown)
3 Left " " " " T1L
3 Right " " " " T2R
3 Left " " " " T2L
4 " " " " T3R
4 " " " " T3L
Omit Lat. Conn. as noted.
All Purlin Clips 1L 3 1/2 x 3 1/2 x 5/16 x 0'-10"
Omit on T3R & T3L
5 Purlin spaces @ 4'-5" = 22'-1"
4 Purlin spaces @ 4'-6 1/2" = 18'-2"
20'-2 1/4" c. to c. Holes
20'-10" c. to c. Holes
45'-8 3/8" c. to c.
Same as member bC
10 Verticals Req. Mark Ee
10 Bot. Chords Req. Mark cc
81'-9" c. to c.
22'-4 5/16"
27'-6 3/8"
Shoe Pl. 12" x 1/2" x 1'-0"
13/16" x 2" Slotted holes in T1L, T2R & T3L
13/16" Round " " T1R, T2L & T3R
For Anchor Bolts.
Lat. Pl. 7 1/2" x 1/4" x 0'-10"
Pin Hole 1 13/16"
Omit on Segments T3R & T3L
Make other side of splice plate symmetrical with this on T3R & T3L
Pl. 23" x 1/4" x 2'-8 1/2"
Pinhole 1 13/16"
22'-9 9/16" c. to c. holes
29'-2" c. to c. holes

Fig. 36.

CHAPTER V.

PLATE GIRDER BRIDGES.

42. Construction and Uses. A plate girder is a built up I-beam. It consists of a single web plate[1] and two flanges (top and bottom) riveted together. Each flange may be composed of two angles, two angles and one or more cover plates, two angles with side and cover plates or, in very heavy girders, four angles with side and cover plates in various combinations. Fig. 37 shows some common flange sections. Types (e) and (f) are frequently used for crane girders where a load is applied along the edges of the flange.

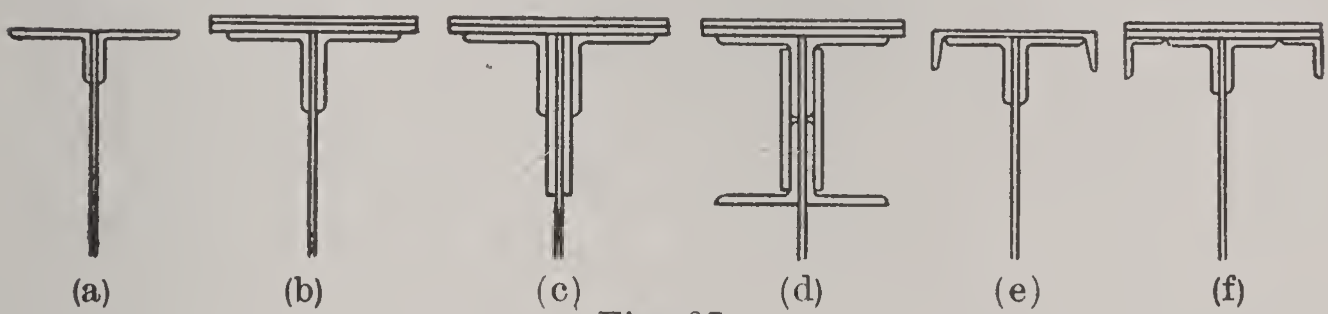


Fig. 37.

Plate girders are used in buildings and bridges where something larger than a rolled I-beam or I-beam girder is required. In buildings they are used for floor and crane girders and in bridges, for stringers and floor beams. and for the girders of plate girder bridges.

Plate girder bridges are seldom built for spans of more than 100 ft., though some have been built over 130 feet long. The railways use them almost exclusively for spans from 30 ft. to 100 ft., when steel bridges are used, and many of the better class of highway bridges of these lengths are plate girders.

A plate girder bridge is usually considered to be the most durable kind of metal bridge.

43. Stresses in Girders.[2] A plate girder is treated as a solid beam and the stresses are investigated by the method of

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[1] When two web plates are used a few inches apart, it is called a box girder."

[2] See Heller's Stresses in Structures," Art. 75, page 111.

sections,[1] the stresses acting upon a cross section being the ones usually found.

The loads, including the weight of the girder itself, which a girder carries, together with the reactions produced by them, are usually a series of forces parallel to the cross section of the girder, and are equivalent to a resultant shear at the section and a couple; these are held in equilibrium by a shearing stress and a couple acting in opposite directions *at the section.* This is illustrated in Fig. 38, in which R at c is the resultant of all forces to the left of the section mn and is equivalent to the shear R at the section and the couple whose moment is Ra and is called the *bending moment.* These are resisted by a *shearing stress* S, equal to R and a moment of a couple $Fd = Ra$. Since the bending moment is the moment of a couple, the moment of the bending stresses must also be that of a couple. The forces F are the *resultants* of the tensile and compressive forces acting on the section, whose intensities vary uniformly from zero at the neutral axis to a maximum intensity at the top and bottom. Since the greater part of the area of the cross section is in the flanges and the intensities are greatest at the top and bottom, the resultants, F, come near the top and bottom of the girder, making d large. Fd is the *moment of resistance,* and it is well to remember that it is equivalent to the moment of a couple, and that *both the web and flanges resist bending.*

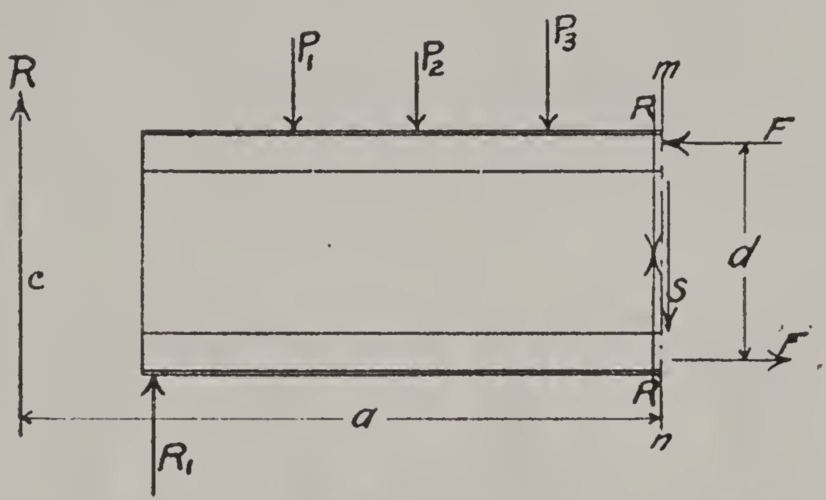

Fig. 38.[2]

44. The Web. It is not known just how the shearing stresses are distributed over an I cross section. We know that their intensities must be zero at the upper and lower edges of

[1] See Heller's "Stresses in Structures," Art. 64, page 85.
[2] See Heller's "Stresses in Structures," Fig. 65, page 86.

the girder, and are a maximum at the neutral axis.[1] The law of variation between these extremes depends upon the cross section. For an I cross section, the usual assumption that the shearing stress is uniformly distributed over the area of the web *only,* will give an intensity of shearing stress which will usually be greater than the actual maximum.[2] The flanges form a large part of the cross section and must carry considerable shear.

It is therefore *always assumed* that the web carries *all of the shear*[3] and that its intensity is uniform.

$$A_w = ht = \frac{S_{max}}{s_s} \quad \ldots\ldots \ \ldots\ldots (1)^4$$

Equation (1) will determine the minimum area of web permissable. Its thickness is never made less than ¼ inch and seldom less than ⅜ inch. It must be made thick enough to give sufficient bearing for the rivets which connect the flanges to it, and this consideration frequently determines its thickness.[5]

The depth, h, is determined by considerations of economy as explained in Art. 46, or by local conditions.

When there are splices in the web, it is not strictly correct to take the gross area as effective in resisting shear. It may be assumed that the rivets of the first row in the splice take up one half of their proportion of the shear on one side of the plane through the center of the row. (4) Then the net section of the web through this row should be sufficient to take the balance of the shear. This would make equation (1) read as follows:

$$\frac{S_{max} - \frac{1}{2}\ \textit{Value of Rivets in Row}}{s_s} = (h - \textit{holes})\,t \ldots\ldots\ldots\ldots (2)$$

Formula (2) is not used in practice, and the difference in the result by the two is usually negligible.

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[1]For a discussion of this subject on the assumption that a sudden change in width of the cross section has no effect upon the distribution of the shearing stress. see Johnson's "Modern Framed Structures," Chapter VIII, Art. 130, page 145.

See also Rankine's "Applied Mechanics," Art. 309, page 338.

[2] See Heller's "Stresses in Structures," Art. 71, page 105.

[3] When the flanges are inclined, they carry a part of the shear.

[4] See Heller's "Stresses in Structures," Eq. 33, page 111.

[5] See an article by C. H. Wood in Eng. News, Aug. 6, 1908.

The web resists considerable bending moment, as will be seen if the formula $M=\frac{sI}{v}$ ....(3)[1] is considered. If, for example, the moment of resistance of the web is $\frac{1}{7}$ of the total moment of resistance of the cross section, the web will resist $\frac{1}{7}$ of the total bending moment and the flanges will resist $\frac{6}{7}$ of it. The resultant of the two kinds of stresses in the web is never calculated, but to compensate for this, it is often assumed that the flanges take all the bending stresses, which, of course, has the effect of making them larger and thus reducing the stress in the web. But if it be remembered that the shear is *not* a maximum where the bending is, it will be seen that, theoretictally, this increase of flange section is not necessary.

45. The Flanges. The bending stresses in a girder may be provided for by making the cross section such that the extreme fiber stress, given by equation (3) will not exceed the maximum allowed unit. This, however, involves much labor, as there are no complete tables of section moduli of plate girders as there are of I-beams, and the solution, involving the two unknown quantities I and v, must be by trial.

When the flanges are alike, as they usually are, the solution is very much simplified by making two assumptions:[2]

"1. The stresses in the flanges (tension and compression) are uniformly distributed over their areas and their resultants, (F, Fig. 38) therefore, act at the center of gravity of these areas.

"2. That the depth of the web h, may be set equal to d, the distance between the centers of gravity of the flanges."

Granting these, it is easily shown that the moment of resistance of the web is equal to the moment of resistance of 1/6 of the web area, concentrated at the center of gravity of each of the flanges.

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[1]For derivation see Heller's "Stresses in Structures," Art. 66 page 89.

[2]See Heller's "Stresses in Structures," Art. 75, page 111, for a complete discussion.

Then we have

$$\textit{Equivalent Flange Stress} = \frac{M}{d} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

$$\textit{Equivalent Flange Area} = \frac{\textit{Equiv. Flg. Stress}}{s_w} \quad \ldots\ldots\ldots (5)$$

Reqd. Flange Area proper (Net Area one Flange)

$$= \textit{Equiv. Flg. Area} - \tfrac{1}{6}A_w \ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

If, for any reason, there are vertical lines of rivet holes in the web, its moment of resistance is decreased, and this is sometimes taken into account by modifying Eq. (6) as given in equation (7) below,[1]

Reqd. Flange Area proper (Net Area one Flange)

$$= \textit{Equiv. Flg. Area} - \tfrac{1}{8}A_w \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

The effect of the rivet holes on the moment of resistance of the web may be easily calculated.

Some specifications require that all of the bending stresses shall be considered as being resisted by the flanges, in which case the equivalent flange area as given by equation (5) becomes the required net flange area proper.

Since d, the *effective depth*, cannot be calculated until the flanges are known, an approximate value must be used on the first trial. Two or three trials will usually give a flange which is practically exact.

In equation (5) the working stress for the tension flange is used, thus giving the required *net* area of that flange. (See Art. 11 for allowance to be made for rivet holes.) The top flange is usually made the same as the bottom flange (gross areas alike) but it must be held so that it will not buckle sidewise.[2] (See Art. 51 and 52.)

46. Economic Depth.[3] The most economical depth of a plate girder is usually the least weight depth. It depends upon a number of conditions and may be easily calculated, theoretically, when these conditions are known. The calculated, economic

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[1]See Specifications of the "American Railway Engineering and Maintenance of Way Association." for Steel Railroad Bridges, 1906, Art. 27.

[2]See Spec. of the Am. Ry. Eng. and M. of W. Assoc., Arts. 28 and 78. Also Cooper's Spec. for Steel Railway Bridges, 1906, Art. 79.

[3]See Johnson's "Modern Framed Structures," Art. 285, page 332.

depth is seldom used exactly, on account of local conditions and practical limitations, and is to be regarded merely as a general guide. A variation in depth of as much as 10% or 15% will usually not change the total weight of the girder appreciably.

Formulas will now be deduced for the economic depth for the following three conditions as to flange section:

(a) When 1/6 of the web area is considered in each flange.

(b) When 1/8 of the web area is considered in each flange.

(c) When none of the web area is regarded as flange area.

The girder will be assumed of constant cross section from end to end.

(*a*) *When 1/6 of the web area is regarded as flange area.*

The gross area of the cross section of the girder $=ht+2A_F+$ *rivet holes.* From equations (4) and (6), $A_F=\frac{M}{ds_t}-\frac{1}{6}dt$ and then we have, setting $h=d$,

$$A=dt+\frac{2M}{ds_t}-\tfrac{1}{3}dt+\textit{rivet holes.}$$

The gross area of the flanges may be taken as 15% greater than the net area, which gives:

$$A=0.617dt+\frac{2.3M}{ds_t}$$

As the weight varies directly with the cross section, for a least weight depth we may differentiate this expression with respect to d and set the first derivative equal to zero.

$$\frac{dA}{dd}=0.617t-\frac{2.3M}{s_t\,d^2}=0$$

from which $$d^2=\frac{2.3M}{0.617s_t\,t}\text{ and}$$

$$d=1.93\sqrt{\frac{M}{s_t\,t}}\quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\quad(8)$$

(*b*) *When 1/8 the web is taken as flange area,* equation (8) becomes

$$d=1.80\sqrt{\frac{M}{s_t\,t}}\quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\quad(9)$$

(*c*) *When no web is considered as flange area,* equation (8) becomes

$$d=1.52\sqrt{\frac{M}{s_t\,t}}\quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\quad(10)$$

When the flange section is not constant for the entire length of the girder, the economic depth will be somewhat less than that given by the above formulas, and will vary with the proportion of cover plates, stiffeners, and web splices. The following equation will give the least weight depth as close as a general formula can give it.

$$d=\sqrt{\frac{M}{s_t\,t}} \qquad (11)$$

47. Stiffeners. The lines of maximum compression in a plate girder web, cross the neutral axis at an angle of 45° and extend downward toward the supports from the middle.[1] The tendency of the web plate to buckle under these compressive stresses is, in part, resisted by the equal tensile stresses at right angles to them. Just what the resulting effect on the web is, is not well understood, but when the ratio of the depth of the web to its thickness is great (exceeds about 50 or 60) it must be stiffened. Of course the requirement of stiffeners depends upon the amount of shear at the point.[2]

Stiffeners are placed vertically on account of ease of manufacture. They would, perhaps, serve their purpose better if placed parallel to the line of the compressive stresses, but if placed vertically and not more than the depth of the girder apart, or 5 or 6 feet for deep girders, they will prevent any buckling of the web.

There is no rational method of determining the size of these stiffeners. Some specifications[2] give column formulas for this but there is no rational basis for it. Practice only determines their size and spacing.

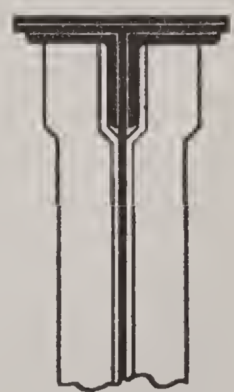

Fig. 39.

Sometimes fillers are put under the stiffeners, between the flange angles, and sometimes the stiffeners are "off set" or "crimped" over the flange angles as shown in Fig. 39. Fillers should be used under stiffeners bearing concentrated loads, or where there is anything connecting to the girder by means of the stiffener.

[1] See Heller's "Stresses in Structures," Art. 73, page 109.
Also see Johnson's "Modern Framed Structures," Art. 130, page 147.

[2] See Cooper's Specifications for Steel Railway Bridges, 1906, Art. 47.
See also Spec. of the Am. Ry. Eng. and M. of W. Assoc., 1906, Art. 77.

Stiffeners should be placed at the ends of a girder, to transmit the end reaction from the web, and at all points of concentrated loading. Stiffeners should bear tightly against the horizontal legs of the flange angles at all points of concentrated loading, as the load must be transmitted to the stiffener by direct bearing upon its end, and from the stiffener, by means of rivets, to the web.

The outstanding leg of the stiffener should not project beyond the edge of the flange angle, and the other leg need only be large enough for the rivets.

48. Web Splices. For small girders, such as stringers and floor beams, the web plates can usually be obtained from the mills in one piece, and no web splices are necessary. When, however, the size of the girder is increased, the web plates cannot be obtained in single lengths and must be spliced.[1]

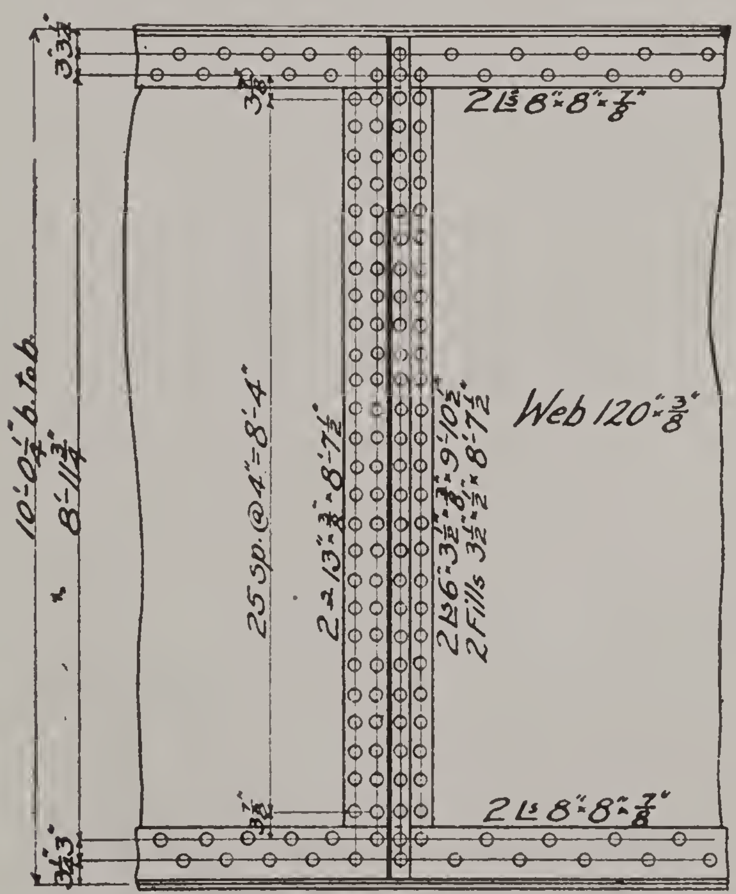

Fig. 40.

The stresses carried by the web at the point must be transferred through the splice plates from one web plate to the other. If no bending moment is regarded as being carried by the web plate, only the shear has to be provided for, and the calculation of the rivets is a very simple matter. In this case a splice similar to that shown in Fig. 40 is used. The rivets are usually not spaced over 5 inches apart, and usually two rows are used on each side of the splice, although this often gives an excess of rivets.

[1]For maximum sizes of plates which may be obtained, see Cambria, page 31. These limits vary considerably with different mills.

Two splice plates should always be used and their combined thickness should be greater than the thickness of the web. A pair of stiffeners is always placed over the splice.

When a part of the bending moment is regarded as being carried by the web, the web splice must be designed to provide for this stress in addition to the shear. The simplest form of web splice to calculate, in this case, is that shown in Fig. 48, in which the plates *FG*, near the flanges, are *assumed* to take care of the bending moment in the web, and the vertical plates *HK*, are assumed to take all the shear. These assumptions give an excess of plate and of rivets, but a rigid calculation would make a splice of practically the same cost. See Art. 52 for the design of such a splice.

In order that the allowed unit stress in the flange proper may not be exceeded, it is necessary to reduce the allowed unit stress in the splice plates *FG*, in proportion to their distance from the neutral axis. The entire solution must, in any case, be by trial.

The form of splice shown in Fig. 40 may be used in place of that of Fig. 48, but the rivets must be calculated so that the resultant of the horizontal and vertical stresses (due to moment and shear) in the outermost rivets will not exceed the allowed stress on a rivet. (12) This would also be the exact method of calculating the rivets in Fig. 48. In Fig. 40 the splice plates act as a beam 8′—7½″ deep to carry the web bending moment, and the extreme fiber stress in them must not exceed the unit stress in the girder at an equal distance from the neutral axis. If the rivets through the web in the outer rows have a small pitch, one-eighth of the web area may not be available as flange area.

49. Flange Riveting The stress in the flange of a girder, at the end, is zero, and it increases to a maximum somewhere between the supports (for a girder supported at the ends). The increase of the flange stress is due to the addition of the horizontal shears,[1] and the rivets connecting the flange to the web, at any point, must be sufficient to transmit this horizontal increment.

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[1]See Heller's "Stresses in Structures," Arts. 17 and 70.

Fig. 41 shows any part of a girder, supported in any manner. M and S are the known moment and shear at the section AB, then

$$M_x = M + S(x-m) - P_1(x-a) - P_2(x-b) - P_3(x-c)$$

Differentiating with respect to x, $\frac{dM_x}{dx}$ will give the rate of increase of the moment along the girder.

$$\frac{dM_x}{dx} = S - P_1 - P_2 - P_3 = S_x$$

The flange stress at any point is $\frac{M_x}{d}$ and, therefore, the rate of increase of the flange stress will be $\frac{dM_x}{dx} \div d = \frac{S_x}{d}$ ..... (**12**)

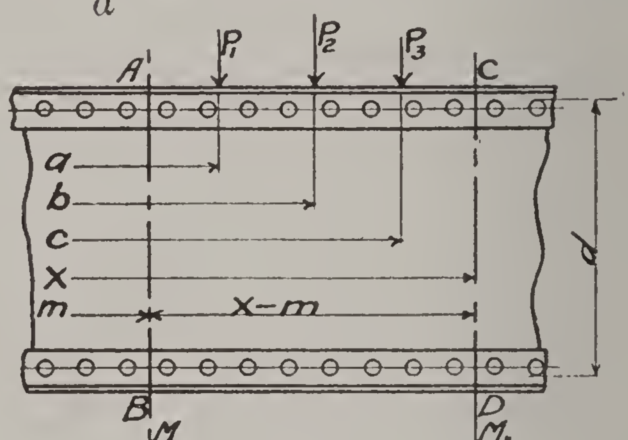

Fig. 41.

or, in words, the increase of the flange stress per inch will be equal to the shear at the point divided by the effective depth of the girder in inches and sufficient rivets must be provided, connecting the flanges to the web, to transmit this increment. Then, to obtain the maximum permissable rivet pitch in inches at any point, the value of a rivet must be divided by the increment per inch. (If there is no vertical load on the flange.)

In a girder whose cross section is *constant from end to end*, and in whose design a part of the web has been considered as flange area, the pitch of the rivets may be increased because the part of the flange stress which is carried by the web does not have to be transmitted by the rivets. Since the flange stresses are directly proportional to the flange areas, we have from equation (12) when one-sixth of the web is regarded as flange area,

Increment of stress in flange proper $= \frac{S_x}{d} \times \frac{A_F}{A_F + \frac{1}{6}A_W}$ ...... (13)

or when one eighth of the web is regarded as flange area,

Increment of stress in flange proper $= \frac{S_x}{d} \times \frac{A_F}{A_F + \frac{1}{8}A_W}$ .... (14)

In a girder with cover plates which do not extend the full length of the girder, and in whose design a part of the web has been regarded as flange area, equation (13) or (14) may be used for that portion of the girder at the ends, whose cross section is constant, but when, passing toward the middle, the first increase in flange section is made by the addition of a cover plate, the flange angles and the part of the web considered as flange area have already received their maximum allowed stress (See Fig. 47) and, therefore, all of the flange stress increment must go into the cover plate and enough rivets to transmit it must be provided, both through the angles and web, and through the cover plate and angles.

So far we have considered only the flange stress increments as affecting the pitch of the rivets. If there are any loads resting upon the flange of the girder, they must be transmitted to the web, and since the web plate is not flush with the backs of the angles, the rivets in the flange must perform this duty unless the load is carried directly by stiffeners.

The top flange of a deck plate girder is a case of this kind, and the resultant of the two stresses (vertical and horizontal) on a rivet must not exceed the allowed rivet value.

Usually only three or four different groups of rivet pitches are used in the half length of a girder. The pitch need be figured only at three or four points and then a curve can be drawn through these, which will give the pitch at any other point with sufficient accuracy. See Fig. 42 and Fig. 47.

The riveting in the two flanges is always made the same when possible, although this gives an excess in the bottom flange.

Sometimes the pitch of rivets in the flange angles will determine the width of the vertical leg of the angles which may be used. For instance, if $2Ls$ 4″x4″ would answer for the flange of a stringer, and it was found that the pitch of rivets required was less than the minimum allowed in a single line, (7) an angle with a vertical leg wide enough for two gage lines would have to be used.

Sometimes the thickness of the web plate will have to be increased over that which would be required to resist the shear, in order to provide sufficient bearing area for the flange rivets, so that they will not have to be spaced closer together than the

minimum allowable pitch. Also the web must be thick enough so that there is sufficient net area, horizontally between the flange rivets, to carry the horizontal increment of flange stress. These considerations sometimes make it necessary to use a thicker web plate at the ends of a girder than in the middle.[1]

50. Flange Splices. There are two conditions which may make it necessary to splice the flange of a plate girder even if the girder is not going to be shipped in more than one piece. These are,

1st. The flange angles may be so heavy that they cannot be obtained in one piece, of the length required. (26)

2nd. When the girder is for a through bridge, frequently the upper corners are rounded, for appearance, and the top flange angles and one cover plate run down the ends of the girder. In this case, the flange is spliced near the ends, so that the long pieces will not have to be handled in the blacksmith shop in heating, bending and annealing.

The flange splices should always be made as near the ends as possible, where the flange stress is least; and the joints in the component parts of the flange should break joints. (29)

The splices are made by means of splice angles cut and ground to fit the inside of the flange angles and by splice plates on the top. The net section of the splice plates and angles does not have to equal the entire net section of the flange cut, unless the entire strength of the cut portion is required at the point of splice. The size of the splice plates and angles and the number of rivets required are determined by the *proportion* of the actual stress at the point, carried by the part cut.

To illustrate the application of the principles set forth in this chapter, the design and methods of calculation for a stringer and a deck plate girder bridge, will now be given.

51. Design of a Stringer. The stringer of a railroad bridge is the simplest form of plate girder. We will assume the following data:

Panel length 27′—0″, Stringers 6′—6″ center to center,
Loading Cooper's Class E 40.

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[1]See an article by C. H. Wood in Eng. News, Aug. 6. 1908.

Specifications Cooper's 1906 for Railway Bridges,
Material medium steel except rivets.

Dead Load. The dead load is estimated and consists of the weight of the floor (rails, ties, guards, fastenings, etc.) and the weight of the stringer itself. Except in special cases, the weight of the floor will not exceed 400 pounds per lineal foot of track, and the specifications §23 will not allow the use of a lesser weight. The weight of the stringer will be taken at 160 pounds per lineal foot. This gives a total dead load per lineal foot of stringer, of 360 pounds.

The dead load moment $= \dfrac{360\times27\times27}{8} = 32{,}800$ ft. lbs.

The live load moment (See Spec. Table I)=344,600 ft. lbs.[1]

Depth. The depth of the stringer must now be decided upon. The economic depth may be determined from equation (10), as §46 of the specifications directs that no part of the web area may be considered as flange area. The value of s_t to use in the formula is determined from §31 of the specifications, and is different for live and dead loads. The dead load moment may be reduced to an equivalent live load moment, in this case by dividing it by 2, and then the live load unit stress may be used with the resultant total moment. This will give a total equivalent moment of 361,000 ft. lbs. Assuming the web to be ⅜″ thick we get

$$d=1.52\sqrt{\frac{361000\times12}{10000\times\frac{3}{8}}}=51.6 \text{ inches.}$$

If there are no local conditions which will limit the depth of the stringer, such as the height from base of rail to masonry, under clearance or depth of floor beam, we can use a 51″ web plate. This will give an area of web of $51\times\frac{3}{8}=19.13$ sq. in.

The maximum shear is as follows:

Live load end shear=59,300 lbs. (See Spec. Table I)
Dead load end shear= 4,900 lbs.

Total Max. end shear=64,200 lbs.

This will give a maximum unit shear on the web of $\dfrac{64200}{19.13}=$ 3,360 lbs. per sq. in., which is safe.

[1]For method of calculation of maximum moment see Heller's "Stresses in Structures," Art. 134, page 260.

The depth back to back of angles is always made ¼″ more than the depth of the web plate (26) so that the web will not project beyond the angles at any point.[1] An approximate effective depth must now be assumed (45) for determining the required flange. We will take 50.5 inches.

Flanges. The following are the approximate flange stresses:

$$\text{Dead Load} = \frac{32800 \times 12}{50.5} = 7{,}800 \text{ lbs.}$$

$$\text{Live Load} = \frac{344600 \times 12}{50.5} = 81{,}900 \text{ lbs.}$$

Dividing these by their respective unit stresses we get,

$$\text{Approx. Req. D.L. Area} = \frac{7800}{20000} = 0.39 \text{ sq. in.}$$

$$\text{Approx. Req. L.L. Area} = \frac{81900}{10000} = 8.19 \text{ sq. in.}$$

$$\text{Approx. Req. Total Net Area} = 8.58 \text{ sq. in.}$$

$2Ls\ 6'' \times 3\frac{1}{2}'' \times \frac{9}{16}''$ gives $2 \times 5.03 - 2 \times \frac{9}{16} \times 1 = 8.93$ sq. in. Net (using ⅞″ rivets).

The effective depth, using these angles with the long legs horizontal, will be $51.25 - 2 \times 0.86 = 49.53$ inches. This will give the following flange stresses,

$$\text{Dead Load} = \frac{32800 \times 12}{49.53} = 8{,}000 \text{ lbs.}$$

$$\text{Live Load} = \frac{344600 \times 12}{49.53} = 83{,}500 \text{ lbs.}$$

and the required areas will be

$$\text{Dead Load Area} = \frac{8000}{20000} = 0.40 \text{ sq. in.}$$

$$\text{Live Load Area} = \frac{83500}{10000} = 8.35 \text{ sq. in.}$$

$$\text{Total Req. Net Area} = 8.75 \text{ sq. in.}$$

[1]Sometimes on stringers without cover plates, the web is made to project an inch above the top flange angles and the ties are notched for this instead of over the entire flange.

It will be found that the flange section above given is the most economical for this case. The actual net area only exceeds the required by 0.19 sq. in.[1]

We must now determine the rivet pitch in the flanges in order to see if we can get the required number of rivets in a single line (49) without using a less pitch than is allowed. (See Spec. §54). (7)

Flange Riveting. Total Max. End Shear=64,200 lbs.

The horizontal increment of flange stress from equation (12) is $\frac{64200}{49.53}$=1296 lbs. per lineal inch. The top flange also carries the weight of the floor and the live load direct, which must be transmitted to the web through the flange rivets. (49) The dead load on the top flange is 200 lbs. per lineal foot, equal to 17 lbs. per lineal inch. The maximum concentrated live load on any point of the stringer will be one driver or 20,000 pounds, which may be considered as distributed over three ties (See Spec. §15) spaced 14 inches center to center, making the load per inch $\frac{20000}{3 \times 14}$=476 lbs. This makes the total maximum vertical load on the top flange 493 lbs. per lineal inch. The resultant of these vertical and horizontal stresses= $\sqrt{(493)^2+(1296)^2}$= 1387 lbs. per lineal inch.

The value of a rivet in bearing on the web is 3938 pounds. (See Spec. §40, floor system.) The maximum allowed pitch at the ends then will be $\frac{3938}{1387}$= 2.83 inches, which is greater than three diameters of the rivet and is, therefore, allowable in a single line.

The required pitches may be determined in a similar manner at the quarter point and center after the shears at these points have been calculated, and when plotted, as in Fig. 42, we can scale the required pitch at any point. The actual pitches used should come within the curve, as shown by the stepped line.

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[1]In some cases, the bottom laterals of the bridge are connected to the bottom flanges of the stringers, in that case the net section is reduced by an extra hole out of the horizontal leg of one flange angle.

Stiffeners. According to the specifications, §47, the web must be stiffened when the shearing stress per square inch exceeds $10000-75H$, in which H is the ratio of the depth of the web to its thickness. By this formula, the maximum allowed shearing stress on this web, without stiffeners, is a negative quantity, so stiffeners must be used throughout the length, spaced not more than the depth of the girder apart (Spec. §47). We will have to put in six pairs of intermediate stiffeners in order to keep within this limit.

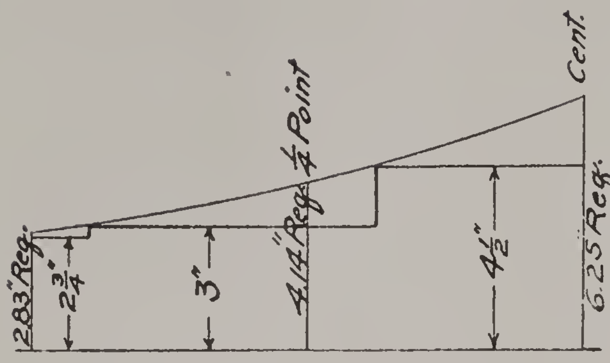

Fig. 42.

The size of the stiffeners must be determined by the column formula given in the specifications §48. The smallest angles which can be used with ⅞″ rivets have 3″ legs (*Cambria*, page 54) and the thinnest metal allowed is ⅜″ thick (Spec. §82), so for the stiffeners we will try 2*Ls* 3″×3″×⅜″. The radius of gyration of the stiffeners, fillers and enclosed web, perpendicular to the web is 1.35 in. $P=10000-45\frac{l}{r}=10000-45\frac{51}{1.35}=8300$ lbs. per sq. in. The gross area of the stiffeners, fillers and enclosed web is 8.72 sq. in., therefore, the stiffeners are good for a shear of $8.72\times8300=72300$ lbs., which is greater than the maximum shear in the girder.

The specifications §79, require that the *compression flanges* of beams and girders shall be stayed against transverse crippling when the length is more than sixteen times the width. In this case the top flanges may have an unsupported length of about 16 feet. They may be held by means of a cross frame at the middle, making the unsupported length 13½ feet, or by means of a subdivided Warren lateral system of single angles between the top flanges, as shown in Fig. 43, making the unsupported length

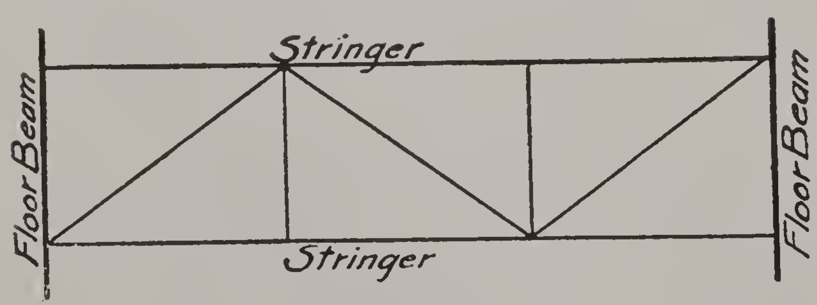

Fig. 43.

of the flange about 9 feet. The size of these angles may be determined by the minimum requirements of the specifications §§82 and 83. A common size is $3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$.

If the track is on a curve, these angles must be made large enough to take care of the centrifugal force.[1]

Estimate of Weight. An estimate of the weight of the stringer will now be made to see how it compares with the weight assumed in the dead load. (160 lbs. per foot)

Flanges 4*Ls* $6'' \times 3\frac{1}{2}'' \times \frac{9}{16}''$ @ 17.1	=	68.4 lbs. per foot.
Web $51'' \times \frac{3}{8}''$	=	65.0 lbs. per foot.
Stiffeners 2*Ls* $3'' \times 3'' \times \frac{3}{8}''$ (Equivalent)	=	22.6 lbs. per foot.
Bracing 1*L* $3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$ (Equivalent)	=	5.0 lbs. per foot.
		161.0 lbs. per foot.
Rivets, say 3%	=	4.8 lbs. per foot.
Total	=	165.8 lbs. per foot.

This is near enough to the weight assumed so that no re-calculation will be necessary.

In all cases the assumed dead load should be checked with the final estimate to make sure none of the actual stresses will exceed those provided for in the design, and also to see if any excess of material has been used over that actually required.

No web splices are necessary, as the web plates can be obtained from the mills in one piece. (See *Cambria*, page 31.)

52. Design of a Deck Plate Girder Bridge. The following data will be assumed:

Span, 103 ft. Extreme (100 ft. c. to c. of bearings).

Loading, Cooper's Class E50.

Specifications, American Railway Engineering and Maintenance of Way Assoc., 1906.

The width center to center of girders should not be less than about $\frac{1}{12}$ the span, and should never be less than six feet for a standard gage track. We will use a width of eight feet center to center.

Floor. The ties will be made 8 inches wide, spaced with six inch openings (Spec. §5). The maximum driver load will

[1]See Heller's "Stresses in Structures," Art. 166, page 307.

be that due to the special loading (Spec. §7) and will be for one rail $\frac{5}{4}\times\frac{50000}{2}=31{,}200$ lbs. This is assumed to be distributed over three ties (Spec. §5), making a pair of loads of 10,400 lbs. each on each tie, besides the dead load. To this must be added 100% of the live load for impact (Spec. §5) and the dead load may be taken at 200 lbs. at each rail for each tie, making a total of 21,000 pounds at each rail. The maximum moment on the tie will be $1\frac{1}{2}\times21000=31{,}500$ ft. lbs. See Fig. 44. Substituting in the formula $M=\frac{sI}{v}$ (Equation 3, Art. 44) we can solve for the depth of the tie directly.

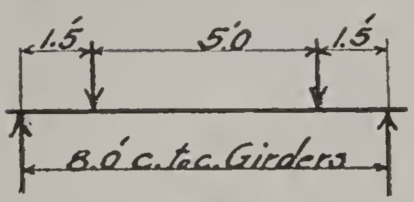

Fig. 44.

$$31500\times12=\frac{2000\times\frac{8d^3}{12}}{\frac{1}{2}d} \text{ from which } d^2=141.75 \quad d=11.9''$$

The ties, therefore, will be $8''\times12''\times11$ ft. long, spaced 14″ center to center.

Dead Load. The weight of the floor can now be calculated (Spec. §6).

Ties $8''\times12''\times11'=396$ lbs. each $\frac{396\times12}{14}=340$ lbs. per lin. ft. of Br.

Guards $2-6''\times8''$ $8\times4\frac{1}{2}=36$ lbs. per lin. ft. of Br.

Rails and fastenings $=150$ lbs. per lin. ft. of Br.

Total weight of floor $=526$ lbs. per lin. ft. of Br.

The weight of the steel work may be estimated by comparison with similar structures, of which the weights are known, or may be approximately determined from an empirical formula of the form

$$w=aL+b^1 \tag{15}$$

In the above formula "b" represents that part of the metal work, the weight of which does not vary appreciably with a change in span length, and may be taken at about 200 lbs. in the present example, and "a" we will assume as 12.5.

[1]See Heller's "Stresses in Structures," Art. 118, page 219.
See also Johnson's "Modern Framed Structures," Art. 62, page 43.

The weight of the steel work$=12.5L+200$
$=1450$ lbs. per lin. ft. of Br.
The weight of the floor, from above$=526$ lbs. per lin. ft. of Br.

Total dead load$=1976$ lbs. per lin. ft. of Br.

Stresses. (43) The maximum live load moments and shears should be calculated at several points in the girder. Then a curve can be drawn through these values when plotted, which will represent the values at all points sufficiently close.

The maximum moment (near the center) and maximum shear (at the end) in the girder, should be calculated from the actual wheel loads[1], and then the moments and shears at the other points may be calculated from equivalent uniform loads[2] derived from this maximum moment and shear.

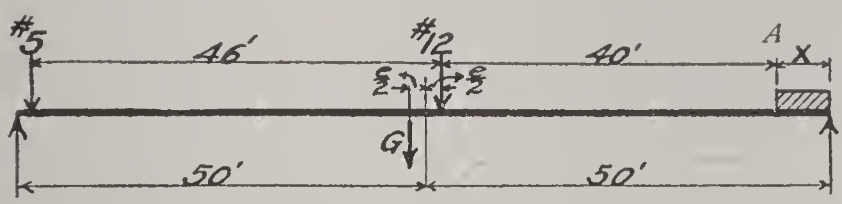

Fig. 45.

The maximum moment will occur with wheel 12 near the center of the span (see Fig. 45) and the center of gravity of all the loads on the span must be as far on one side of the center as wheel 12 is on the other. Then from Fig. 45 we can write the following equations:

$$\frac{M_A + W_A x + \dfrac{wx^2}{2}}{W_A + wx} = 50 + \frac{e}{2}$$

$$40 + x = 50 - \frac{e}{2}$$

M_A =Moment about A of all loads to the left of A, on the span.

W_A =Sum of all loads to the left of A, on the span.

w =Uniform load per lineal foot.

Substituting in the above equations and solving for e and x, we get $e=0.2$ ft. $x=9.9$ ft.

Having determined the position of the loads, the calculation of the moment is a simple matter. The work may be greatly facilitated by the use of a moment table if one is available.

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[1]See Heller's "Stresses in Structures," Art. 134, page 260.

[2]See Heller's "Stresses in Structures," Art. 144, page 269.

The maximum live load moment in the girder is 4,025,000 ft. lbs.

The maximum live load end shear will occur with wheel 2 at the end of the girder, and is 187,500 lbs.

The equivalent uniform load for moments is determined by setting the maximum live load moment equal to $\frac{wL^2}{8}$ and solving for w.

$$4{,}025{,}000 = \frac{w \times 10000}{8}$$

$w = 3220$ lbs. per lin. foot of girder.

The equivalent uniform load for shears is obtained by setting the maximum live load end shear equal to $\frac{wL}{2}$ and solving for w.

$$187{,}500 = \frac{w \times 100}{2}$$

$w = 3750$ lbs. per lin. foot of girder.

From these equivalent uniform loads, the stress at any point can be obtained with sufficient accuracy.

The uniform load moment varies as the ordinates of a parabola, and can be scaled from a diagram drawn as shown in Fig. 47.

The shears will now be figured from the equivalent uniform load at points $16\frac{2}{3}$ ft., $33\frac{1}{3}$ ft., and 50 ft. from the end, (selected on account of the location of the web splices as determined later.)

For location of points A, B, C and D see stress sheet, Fig. 53.

Live load shear at $A = \frac{3750 \times 100_2}{100 \times 2} = 187.500$ lbs.

Live load shear at $B = \frac{3750 \times 83.3^2}{100 \times 2} = 130{,}200$ lbs.

Live load shear at $C = \frac{3750 \times 66.7^2}{100 \times 2} = 83{,}300$ lbs.

Live load shear at $D = \frac{3750 \times 50^2}{100 \times 2} = 46{,}900$ lbs.[1]

[1]The live load shear at D calculated from the actual wheel loads is 49,200 lbs., or 4.9% greater than that given by the equivalent uniform load.

To each of the stresses thus far determined, must be added the impact stress as determined by the formula given in the specifications §9, $I = S\dfrac{300}{L+300}$

Summary of Stresses.

Maximum Live Load Moment$=$4,025,000 ft. lbs.

$\text{Impact} = \dfrac{4025000 \times 300}{100 + 300} = 3{,}018{,}800$ ft. lbs.

$\text{Dead Load} = \dfrac{1976 \times 100 \times 100}{8 \times 2} = 1{,}235{,}000$ ft. lbs.

Total Maximum Moment$=$8,278,800 ft. lbs.

Live Load End Shear$=$187,500 lbs.

$\text{Impact} = \dfrac{187500 \times 300}{100 + 300} = 140{,}600$ lbs.

$\text{Dead Load} = \dfrac{1976 \times 100}{4} = 49{,}400$ lbs.

Total End Shear $=$377,500 lbs.

Live Load Shear at $B$$=$130,200 lbs.

$\text{Impact} = \dfrac{130200 \times 300}{83\frac{1}{3} + 300} = 102{,}000$ lbs.

$\text{D.L.} = 49400 - \frac{1}{3} \times 49400 = 32{,}900$ lbs.

Total Shear at B $=$265,100 lbs.

Live Load Shear at $C$$=$ 83,300 lbs.

$\text{Impact} = \dfrac{83300 \times 300}{66\frac{2}{3} + 300} = 68{,}200$ lbs.

$\text{D.L.} = 49400 - \frac{2}{3} \times 49400 = 16{,}500$ lbs.

Total Shear at C $=$168,000 lbs.

Live Load Shear at $D$$=$ 46,900 lbs.

$\text{Impact} = \dfrac{46900 \times 300}{50 + 300} = 40{,}200$ lbs.

Dead Load Shear $=$ 000 lbs.

Total Shear at D $=$ 87,100 lbs.

Depth. The economic depth (46) can now be figured from equation (11), assuming the web to be 3/8 inches thick.

$$d = \sqrt{\frac{12 \times 8278800}{16000 \times \frac{3}{8}}} = 128.7 \text{ inches.}$$

The cost per pound of plates increases very rapidly as the width increases, after 100 inches is passed, until at 130 inches wide the cost has reached about 40% excess per pound over the cost of plates 100 inches wide and less. Also it must be remembered that pieces over about ten feet deep cannot be shipped on many roads, (31) so we will make the web of the girder only 120 inches deep, instead of the depth given by the formula. This is probably more nearly the least cost depth, than that given by the formula, in this case.

Web. The web plate will have to take the shear (44) without exceeding a unit stress of 10,000 lbs. per sq. in. on the gross area (See specifications §18). The required area will be $\frac{377500}{10000}$ $=37.75$ sq. in. $\frac{37.75}{120}=0.315$ inches required thickness, so we can use a web plate $120''\text{x}\frac{3}{8}''$.

Flanges. According to the specifications §27, one-eighth of the gross web area may be regarded as flange area. (45). For an approximate effective depth 10 ft. will be assumed. This gives a flange stress of 827,900 lbs. The approximate required area will be $\frac{827900}{16000}=51.74$ sq. in. net.

The following flange section will be tried:

	Sq. in. gross	Sq. in. Net.
$\frac{1}{8}$ Web$=\frac{1}{8}\times 45.00=$	5.62	5.62
$2Ls\ 8''\times 8''\times\frac{7}{8}''$	$=26.48$	$26.48-6\times 1\times\frac{7}{8}=21.23$
$1-20''\times\frac{1}{2}''$	$=10.00$	$10.00-2\times 1\times\frac{1}{2}=9.00$
$1-20''\times\frac{1}{2}''$	$=10.00$	$10.00-2\times 1\times\frac{1}{2}=9.00$
$1-20''\times\frac{7}{16}''$	$=8.75$	$8.75-2\times 1\times\frac{7}{16}=7.87$
	Total gross$=60.85$	Total net$=52.72$

The net areas here have been figured with two rivet holes out of the vertical legs of the angles and one out of each horizontal leg. (11) To obtain this as a minimum net area the pitch of the rivets in the cover plates must never be less than about $2\frac{3}{4}$ inches.

The location of the center of gravity of this flange is 0.74 inches from the backs of the angles as shown in Fig. 46, making the actual effective depth $120.25-2\times 0.74=118.77$ inches.

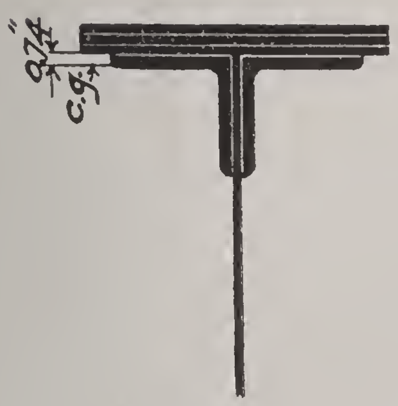


Fig. 46.

The actual flange stress then is $\frac{8278800 \times 12}{118.77}$ $=836,500$ lbs., making the required area $=\frac{836500}{16000}=52.28$ sq. in., so the above flange section will answer.

Length of Flange Plates. The required lengths of the flange plates may be calculated from the equation of the parabola, in this case, as we are using the equivalent uniform load for the moments. Or the lengths may be determined graphically by drawing the parabola, as shown in the upper diagram of Fig. 47. The graphic method is nearly always used, as the moment diagram is frequently not a simple curve.

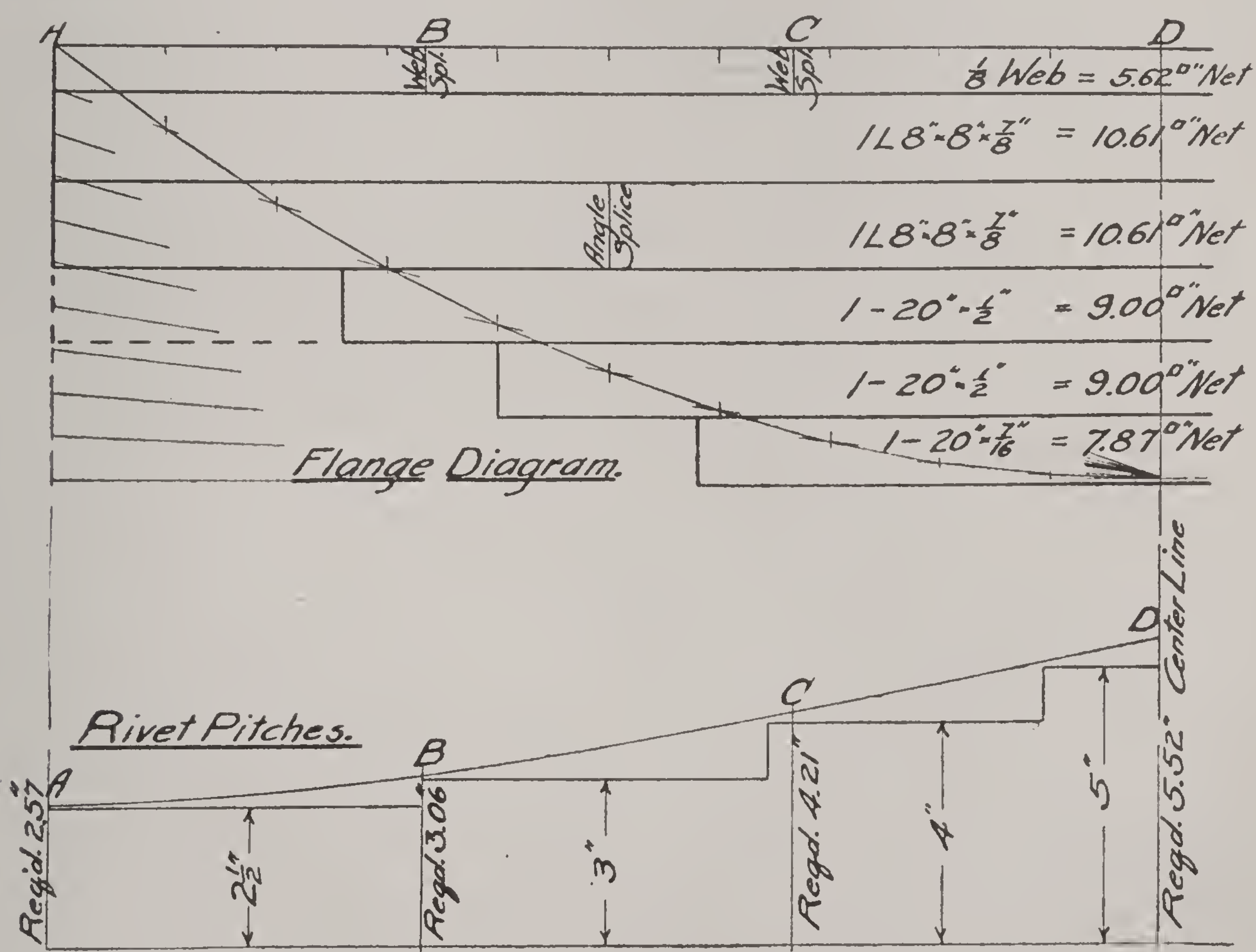


Fig. 47.

As the moments and required flange areas are directly proportional, (when the effective depth is constant) the areas will vary as the ordinates of a parabola also, and it is simpler to lay off areas as ordinates instead of moments. Any convenient scales may be chosen for lengths and areas. In this case the

middle ordinate of the parabola is 52.28 (=Reqd. net flange area) and the curve may be drawn in any one of several ways, the method shown is a simple one.

After the curve of required areas is drawn in, the net areas of the component parts of the flange are measured off on the center line and horizontal lines through these points represent the parts. The required lengths of the various pieces may now be scaled off directly. The flange plates are made 2 or 3 feet longer than the theoretic length in order to provide a few rivets through the plate near the ends so that the strength may begin to be effective where it is required, and also to compensate for the fact that the actual wheel loads give slightly larger moments near the ends than the equivalent uniform load. The effective depth decreases a little toward the ends, owing to the omission of the flange plates, and this will also make the flange stress a little greater there than is given by the parabola.

For a symmetrical girder, only one half the diagram need be drawn.

The top flange plate next to the angles is nearly always run the full length of the girder, to cover the flange angles and to stiffen them near the ends, against the deflection of the ties. (See specifications §76.)

Scaling from the diagram, Fig. 47, the following are the required lengths of the flange plates:

$20'' \times \frac{1}{2}''$—70 ft. use 74 ft.
$20'' \times \frac{1}{2}''$—56 ft. use 60 ft.
$20'' \times \frac{7}{16}''$—38 ft. use 42 ft.

Flange Riveting. (49). The horizontal increment of flange stress at A (See Fig. 53) may be determined from equation (14). As the top flange also carries the direct load of the floor and live load the required pitch will be less for it than for the bottom flange, so the bottom flange pitch need not be figured.

Horizontal increment$=\frac{377500}{116\ 31}\times\frac{36.48}{42.10}=$2812 lbs. per lin. in.

The gross areas are used for proportioning the stress. Note also that the effective depth here is less than at the middle of the girder.

The vertical load from the floor is 263 lbs. per lineal foot and the weight of the flange of the girder here is 127 lbs. per

lineal foot, making a total dead load of

390 lbs. per lineal foot $=$ 33 lbs. per lin. in.

Live load (See Spec. §§ 5 and 29) $= \frac{25000}{42} =$ 595 lbs. per lin. in.

Impact (See Spec. §5) $=100\%$ $=$ 595 lbs. per lin. in.

Total vertical load $=1223$ lbs. per lin. in.

The total resultant stress on the rivets will be

$\sqrt{(2812)^2+(1223)^2}=3066$ lbs. per lineal inch.

Required rivet pitch $=\frac{7876}{3066}=2.57$ inches.

To find the horizontal increment of flange stress at B, we must use equation (12) because the web is here carrying all the bending stress allowed, and all of the increment goes into the flanges proper.

Horizontal increment $=\frac{265100}{117.01}=2266$ lbs. per lineal inch.

Vertical load (same as before) $=1223$ lbs. per lineal inch.

Resultant Stress $=\sqrt{1223^2+2266^2}=2575$ lbs. per lineal inch.

Required rivet pitch $=\frac{7876}{2575}=3.06$ inches.

Horiz. increment at $C=\frac{168000}{118.77}=1415$ lbs. per lineal inch.

Resultant Stress $=\sqrt{1223^2+1415^2}=1870$ lbs. per lineal inch.

Required rivet pitch $=\frac{7876}{1870}=4.21$ inches.

Horiz increment at $D=\frac{87100}{118.77}=$ 733 lbs. per lineal inch.

Resultant Stress $=\sqrt{1223^2+733^2}=1426$ lbs. per lineal inch.

Required rivet pitch $=\frac{7876}{1426}=5.52$ inches.

These rivet pitches are plotted as shown in Fig. 47 and the actual pitches used are made to come within the curve as shown by the stepped line.

The required pitch of rivets through the flange plates is determined by the horizontal increment of flange stress alone. The total shear at the theoretical end of the first flange plate is 276,000 lbs. This gives a horizontal increment of $\frac{276000}{117.01}=2360$

lbs. per lineal inch and a required rivet pitch of $\frac{2\times7216}{2360}=6.1$ inches. As the maximum allowed pitch is 6 inches (See specifications §37) it will not be necessary to calculate the pitch at any other points. The pitch of the rivets in the cover plates must bear some relation to that of the rivets through the vertical legs of the flange angles so that they will not interfere. (29)

Flange Splices. (50). About the maximum length of angle 8″x8″x⅞″ which can be obtained in one piece is ninety feet, therefore the flange angles will have to be spliced. We will splice one angle of each flange about 25 feet from each end of the girder so that both angles of one flange will not be cut at the same point.

At this point the flange angles are carrying the maximum allowed stress, and the total stress in one angle will be $10.61\times16{,}000=169{,}800$ lbs.

To take this stress we will use a splice angle on the inside of the angle spliced and a plate inside of the other angle. The splicing material required, then, will be 1L 8″x8″x $\frac{11}{16}$″ cut down to 7″x7″x $\frac{11}{16}$″ and ground to fit the fillet of the flange angle and one plate 7″x$\frac{11}{16}$″ These will have an available net area of 10.53 sq. in. The length of the angles will have to be sufficient to take enough rivets to transmit 169,800 lbs., and one-third of this must be transmitted to the plate on the side opposite the angle cut. According to the specifications §55, the rivets connecting this plate must be increased 66⅔% over the number required by §18 for the angle in contact with the cut member.

Stress in one leg of splice angle $=\frac{169800}{3}=56{,}600$ lbs.

Rivets required in angle on side next to splice $=\frac{56600}{7216}=7.85$

66⅔% $=5.25$

Rivets required in angle on opposite side $=13.1$

The rivet pitch at the splice may be made 3 inches, which gives us a splice plate 6½ ft. long on the side opposite the splice and an angle 4 ft. long on the side next to the splice.

Stiffeners. (47). The stiffeners must be proportioned according to specifications §§ 16 and 77. The end shear which must be transmitted by the end stiffeners to the abutment is

377,500 lbs. To take this load we will need $\frac{377500}{7876}+50\%=72$ rivets. (See Spec. § 56.)

This number can be put into three pairs of stiffener angles with a single line of rivets in each angle. The stress, then, on each pair of angles will be, $\frac{377500}{3}=125,800$ lbs. The outstanding legs of these end stiffeners must be as wide as the flange angles will allow, so we will try for these 2*Ls* 7″x3½″x½″.

The allowed unit stress is $16000-70\ \frac{60}{4.24}=15000$ lbs. per sq. in.

The required area of one pair of angles$=\frac{125800}{15000}=8.39$ sq. in.

We can, therefore, use for these stiffeners, 2*Ls* 7″x3½″x$\frac{7}{16}$″ whose area is 8.82 sq. in.

The minimum size of angles allowed for the *intermediate stiffeners* is $\frac{120}{30}+2=6$ inches for the outstanding leg. (See Spec. § 77). We will use for these 2*Ls* 6″x3½″x⅜″.

The spacing of the intermediate stiffeners must not exceed the distance "*d*" allowed by the formula in §77 of the specifications.

$$\text{Required spacing at } A=\frac{3/8}{40}\left(12000-\frac{377500}{45}\right)=34 \text{ inches.}$$

$$\text{Required spacing at } B=\frac{3}{320}\left(12000-\frac{265100}{45}\right)=57 \text{ inches.}$$

$$\text{Required spacing at } C=\frac{3}{320}\left(12000-\frac{168000}{45}\right)=77 \text{ inches.}$$

$$\text{Required spacing at } D=\frac{3}{320}\left(12000-\frac{87100}{45}\right)=94 \text{ inches.}$$

Web Splices. (48). The total length of the girder is 103 ft. Plates 120 inches in width and only ⅜ inches thick are not listed in the Cambria hand book, but in the Carnegie shape book they are given and can be obtained up to 220 inches long, or 18′—4″. It will therefore be necessary to splice the web at five points, making it in six pieces. The end sections may be made 18′—2″ long and the intermediate sections each 16′—8″. This will make the spacing of the cross frames uniform.

The maximum bending moment at the first splice *B*, is as follows:

Dead Load = 686000 ft. lbs.
Live Load = 2236000 ft. lbs.
Impact = 1677000 ft. lbs.

Total = 4599000 ft. lbs.

The actual flange area effective at this point is 35.84 sq. in., and therefore the bending moment taken by the web here is $\frac{5.62}{35.84}\times 4{,}599{,}000 = 721{,}000$ ft. lbs. At the splice this moment must be resisted by the splice plates *FG*, and the stress in these plates due to the moment will be $\frac{721000\times 12}{92} = 94{,}000$ lbs.

The maximum allowed unit stress on the extreme fiber of the girder is 16,000 lbs. per sq. in., and the maximum allowed unit stress on the splice plates *FG* will be proportional to their distances from the neutral axis of the girder, or,

$$\frac{46}{60.6}\times 16{,}000 = 12{,}145 \text{ lbs. per sq. in.}$$

and the required area in the splice plates will be $\frac{94000}{12145} = 7.73$ sq. in.

This will require 2 plates 12″x½″ (net area = $12.00 - 4\times 2\times 1\times \frac{1}{2} = 8.00$ sq. in.).

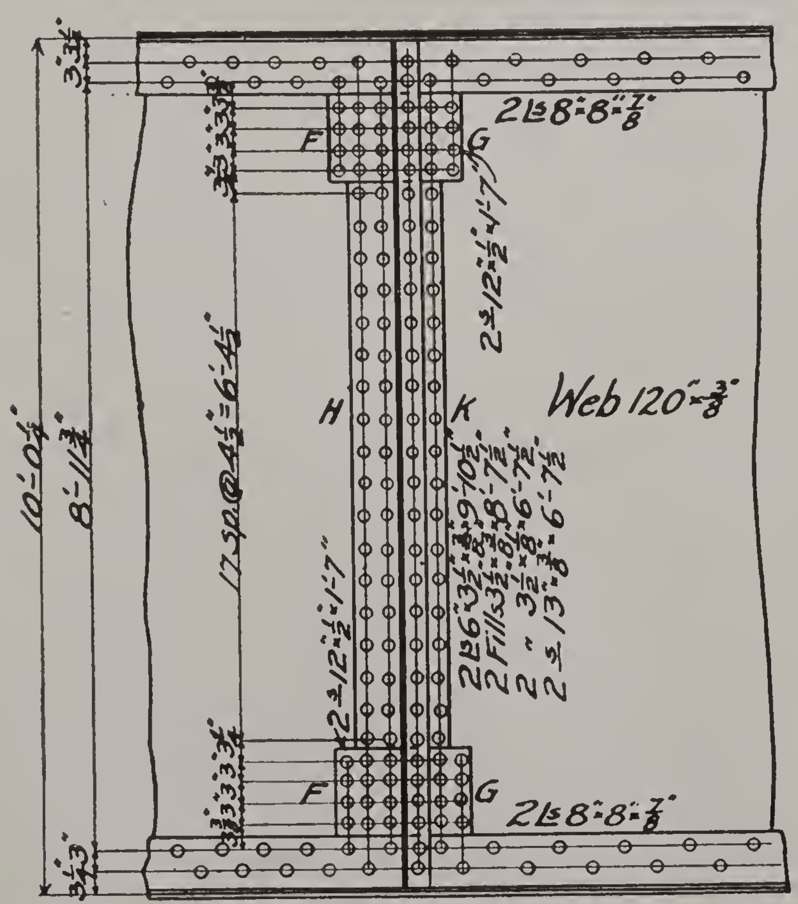


Fig. 48.

The number of rivets on each side of the splice in these plates will be $\frac{94000}{7876} = 12$.

The vertical splice plates to take the shear will require $\frac{265100}{7876} = 34$ rivets on each side. The design shown in Fig. 48 has 36 rivets on each side.

These figures are for the splice at *B*, but usually the same design is used for all the splices.

It will be noted that the maximum shear and maximum moment have been used here as occurring simultaneously. This is on the side of safety, but a rigid solution would not give a splice appreciably smaller.

A splice similar to the one shown in Fig. 40 may be used and calculated as follows. Here the number of rivets must first be assumed and then the stress in them calculated, to see that it does not exceed the allowed units.

The splice, as drawn in Fig. 40, contains 52 rivets on each side, the vertical stress on each rivet, due to shear, is $\frac{265100}{52}=$ 5,100 lbs. The bending moment to be resisted by these rivets is $721{,}000\times12=8{,}652{,}000$ inch pounds. The amount of stress on each rivet due to bending moment will be in direct proportion to its distance from the neutral axis. (12)

Calling the stress on the outermost rivet S, we have:

$4S\times50+4S\times\frac{46}{50}\times46+4S\times\frac{42}{50}\times42+\ldots=8{,}652{,}000$, or using the letter y to represent the distance from the neutral axis to the rivet, in each case, we have:[1]

$$\frac{4\,S}{50}\Sigma y^2=M$$

In this case $\Sigma y^2=11{,}700$ and $S=\frac{8652000\times50}{4\times11700}=9{,}243$ lbs.

The resultant maximum stress on the outer rivet is $\sqrt{5100^2+9243^2}=10{,}560$ lbs., which is in excess of the allowed unit, (7876), and therefore the number of rivets would have to be increased if this type of splice were used in this girder.

The splice plates must be strong enough, when considered as a beam 103½ inches deep, to carry the web's proportion of the bending moment without exceeding a unit stress at the top and bottom, proportional to the distance from the neutral axis, or $\frac{51\ 75}{60.6}\times16{,}000=13{,}680$ lbs. in this case. In figuring the moment of inertia of the plates, the rivet holes should be deducted.

Lateral Bracing.[2] To provide for wind stresses and vibra-

[1]Strictly, these forces are perpendicular to lines drawn from the center of gravity of the entire group of rivets. to each rivet.

[2]See Heller's "Stresses in Structures," Chapter XIV.

tions (See Spec. §10) a lateral system must be put in the span. Sometimes two systems are used, one in the plane of each flange, and sometimes only one is used, in the plane of the top flange, and the forces from the lower flange are transferred to the upper system by means of cross-frames (See Fig. 51) at intervals. Cross-frames are also put in to stiffen the bridge, when two systems of laterals are used. They are usually placed from 15 to 20 feet apart, depending upon the width of the flanges of the girders.

In a deck plate girder bridge, the lateral system is of the Warren, or sub-divided Warren type of truss with an even number of panels, so as to be symmetrical about the center line. The number of panels is so chosen that the laterals will be efficient, that is, so that they will not be inclined at too great an angle with the direction of the wind. Also, the panels must be short enough so that the actual unit stress in the top flange of the girder will not exceed that allowed by the specifications §28.

The actual unit stress in the top flange is $\frac{833600}{60.85}=13,700$ lbs. per sq. in. Equating this to the unit as given in §28 and solving for l, we get $13,700=16,000-200\frac{l}{b}=16,000-10l$ from which $10l=2,300$ and $l=230$ inches$=19'\text{-}2''$.

The unsupported length of the top flange must not exceed this amount.

We will divide the span into 12 panels, using a single system in the plane of the top flange, and put in cross frames at every second panel point. This will make a cross frame fall at each web splice. This is not necessary, but a *stiffener* must be at each cross frame.

As but one system of laterals is to be used, it must be proportioned to carry the entire lateral force.[1] From the specification §10 the load is 200+200+10% of 5000=900 lbs. per lineal foot of girder, and all of this is to be considered as a moving load.

[1]For the calculation of the stresses in lateral systems of bridges having curved track see Heller's "Stresses in Structures," Art. 166, page 304.

The stresses in the laterals will be alternately compression and tension, and will all reverse when the direction of the wind reverses. Laterals, however, are never designed for reversals of stress, (See Spec. §20) so far as the reversal of the wind is concerned, because such reversals would occur only at long intervals.

Since it requires more material to take care of the compression than the tension in a lateral, we are concerned only with the compressive stresses, and choose that direction of the wind which, for any particular lateral, will give compression in it. It will be assumed that the load is all applied at the windward panel points, although the live load is really applied at both girders. This assumption is on the safe side, and simplifies the calculation of stresses.

On account of the cross frames, there are 12 panels on one side and six on the other. When the wind is blowing as indicated in Fig. 49, all of the panel loads will be equal and are $16.67 \times 900 = 15{,}000$ lbs. each. This direction of the wind will give maximum compressive stresses in *BC, DE,* and *FG,* and these will be as follows:

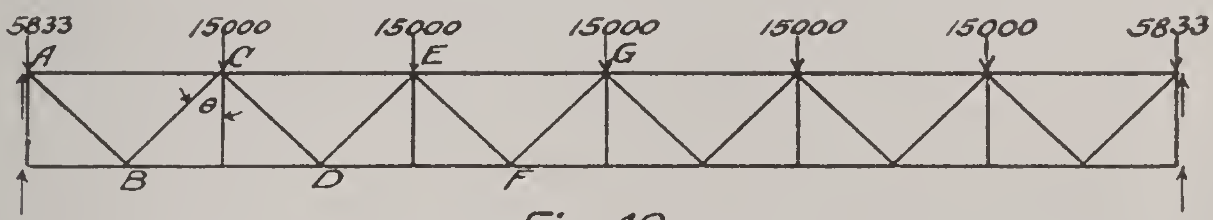


Fig. 49.

$$\text{Sec. } \theta = \frac{11.55}{8}$$

$$BC = 2\tfrac{1}{2} \times 15{,}000 \times \sec.\theta = 54{,}100 \text{ lbs.}$$
$$DE = \tfrac{10}{6} \times 15{,}000 \times \sec.\theta = 36{,}100 \text{ lbs.}$$
$$FG = \tfrac{6}{6} \times 15{,}000 \times \sec.\theta = 21{,}600 \text{ lbs.}$$

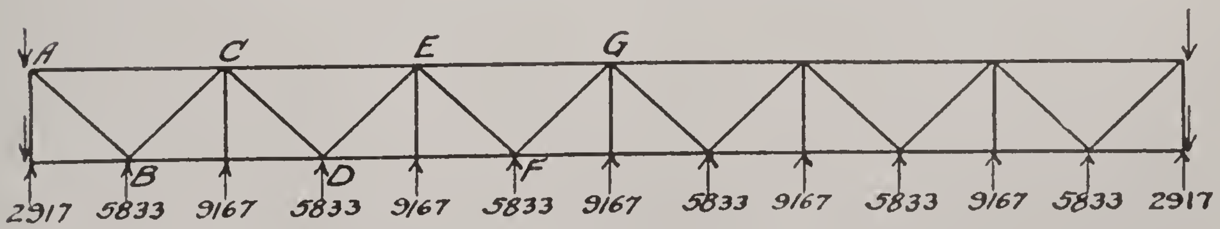


Fig. 50.

When the wind blows in the other direction as shown in Fig. 50, the compressive stresses will occur in *AB, CD* and *EF.* The panel loads on the lateral system for full loading are shown.

$$AB=40{,}420\times sec.\theta=58{,}500 \text{ lbs.}$$
$$CD=27{,}430\times sec.\theta=39{,}600 \text{ lbs.}$$
$$EF=16{,}950\times sec.\theta=24{,}500 \text{ lbs.}$$

According to the specification §23, the unit stress for laterals may be increased 25% over that given for other members, and according to §72 the smallest angle allowed is 3½″x3″x⅜″. It is usual, where possible, to make laterals of single angles. The least radius of gyration of a single angle 3½″x3″x⅜″ is 0.62″. The unsupported length may be taken as the length between edges of flange angles, and in this case will be about 11.55—1.55=10.0 ft.=120 inches.

$16{,}000-70\,\frac{l}{r}=2{,}450$ lbs. per sq. in. Adding 25% gives 3,060 lbs. per sq. in. This makes 1*L* 3½″x3″x⅜″ worth 2.30×3,060=7,040 lbs., which is far less than the least stress in the lateral system.

Trying 1*L* 4″x4″, the least radius of gyration is 0.79″ and the allowed unit stress is 6710. The required area for $FG=\frac{21600}{6710}=3.22$ sq. in.

Use for FG 1*L*4″x4″x $\frac{7}{16}$″. Actual area=3.31 sq. in.

Using a 4″x4″ angle for *EF* would require $\frac{24500}{6710}=3.66$ sq. in. This would require 1*L*4″x4″x½″ which weighs more than 1*L*5″x5″x⅜″, so we will try 1*L*5″x5″. The least radius of gyration=0.98″ and the allowed unit stress in 9,300 lbs. per sq. in. The required area for $EF=\frac{24500}{9300}=2.64$ sq. in.

Use for EF 1*L*5″x5″x⅜″. Actual area=3.61 sq. in.

Using a 5″x5″ angle for *DE* requires $\frac{36100}{9300}=3.89$ sq. in.

Use for DE 1*L*5″x5″x $\frac{7}{16}$″. Actual area=4.19 sq. in.

Using a 5″x5″ angle for *CD* requires $\frac{39600}{9300}=4.26$ sq. in., so we will try 1*L*6″x6″. The least radius of gyration is 1.18″ and the allowed unit stress is 11,110 lbs. per sq. in. The required area for $CD=\frac{39600}{11110}=3.56$ sq. in. *Use for CD* 1*L*6″6″x⅜″. Actual area=4.36 sq. in.

Required area for $BC = \frac{54100}{11110} = 4.87$ sq. in. Use for BC $1L6'' \times 6'' \times \frac{7}{16}''$. Actual area$=5.06$ sq. in.

Required area for $AB = \frac{58500}{11110} = 5.27$ sq. in. Use for AB $1L6'' \times 6'' \times \frac{1}{2}''$. Actual area$=5.75$ sq. in.

The end cross frames must be proportioned to carry all the wind load to the abutment. It is usually considered that half of this goes through each diagonal to the supports, one diagonal being in tension and the other in compression.

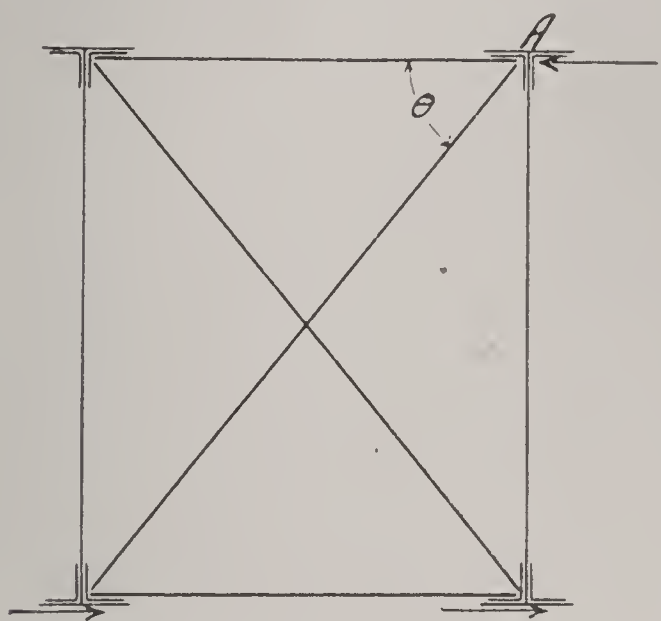


Fig. 51.

The total force acting at the top of the cross frame at A (See Fig. 49) is 43,300 lbs. $Sec.\theta = \frac{12.8}{8}$. Stress in top strut$=21,700$ lbs.

Stress in diagonal$=21,700 sec\theta = 34,600$ lbs. The diagonal in compression is supported at the middle in one direction by the tension diagonal, so an angle having unequal legs will be more economical than an equal legged angle for the diagonals.

Try $1L6'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$. Allowed unit stress is $16000 - 70\frac{150}{1.94} + 25\% = 13,240$ lbs. per sq. in.

Required area$= \frac{34600}{13240} = 2.62$ sq. in. Actual area$=3.43$ sq. in.

As this is larger than necessary we will try $1L5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$. The allowed unit stress is 11,800 lbs. per sq. in. Required area $= \frac{34600}{11800} = 2.93$ sq. in. Actual area$=3.05$ sq. in. *Use for diagonals* $1L5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$.

For the top strut try $1L3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$. Allowed unit stress$=9,000$ lbs. per sq. in. Required area$= \frac{21700}{9000} = 2.41$ sq. in. Actual area$=2.49$ sq. in. This is large enough so *use for top and bottom struts* $1L3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$.

The amount of load transferred by the intermediate cross frames is only 3,333 lbs., so the smallest angle allowed by the specifications will be sufficient. *Use for the intermediate cross frames* 3½″x3″x⅜″ angles.

The number of rivets in the end connections of the lateral members is determined by the single shear value of a rivet. The laterals will be field riveted, so the value of a rivet will be 6,013 lbs.

AB requires $\frac{58500}{6013}=10$ rivets. *DE* requires $\frac{36100}{6013}=6$ rivets.

BC requires $\frac{54100}{6013}=9$ rivets. EF requires $\frac{24500}{6013}=5$ rivets

CD requires $\frac{39600}{6013}=7$ rivets. *FG* requires $\frac{21600}{6013}=4$ rivets.

End cross frame diagonals require $\frac{34600}{7216}=5$ rivets. (Shop)

End cross frame struts require $\frac{21700}{7216}=3$ rivets. (Shop)

The intermediate cross frames will have to have all connections made with 3 rivets each, to comply with the specifications §72.

Shoes. The shoe should be of such design that it will distribute the end reaction evenly over the masonry. For short spans it is customary to simply rivet a plate, not less than ¾ inches thick, under the end of the girder, and allow this to rest on another similar plate resting on the masonry. With this form of shoe the load is applied heaviest at the inner edge of the masonry plate on account of the deflection of the girder. The best results are obtained, especially for long spans, by using hinged bolsters. (See Spec. §61.)

The required bearing on the masonry (assuming sandstone) is $\frac{377500}{400}=944$ sq. in. (See Spec. §19). Using a shoe 3 ft. long, it will require $\frac{944}{36}=26.2$ inches width.

The smallest rollers allowed are 6 inches in diameter (Spec. §58 and §60) and it will require $\frac{377500}{3600}=105$ lineal inches of rollers under each bearing. (See Spec. §19). This will require five rollers each 21 inches long.

The pin must be large enough to properly transmit the shear, and the required area is $\frac{377500}{2 \times 12000} = 15.7$ sq. in. This requires a pin $4\frac{1}{2}$ inches in diameter.

The shoe must be strong enough to distribute the end reaction, as a beam, from the pin, evenly over the masonry, and must also be strong enough laterally to transmit the wind forces to the abutments.

Fig. 52 shows a good design for the shoe.

The estimate of weight can now be made upon forms as described in Art. 19, and if the actual dead load taken from the estimate does not differ enough from the assumed dead load to cause a change in the size of any of the members, the stress sheet may be drawn, as shown in Fig. 53.

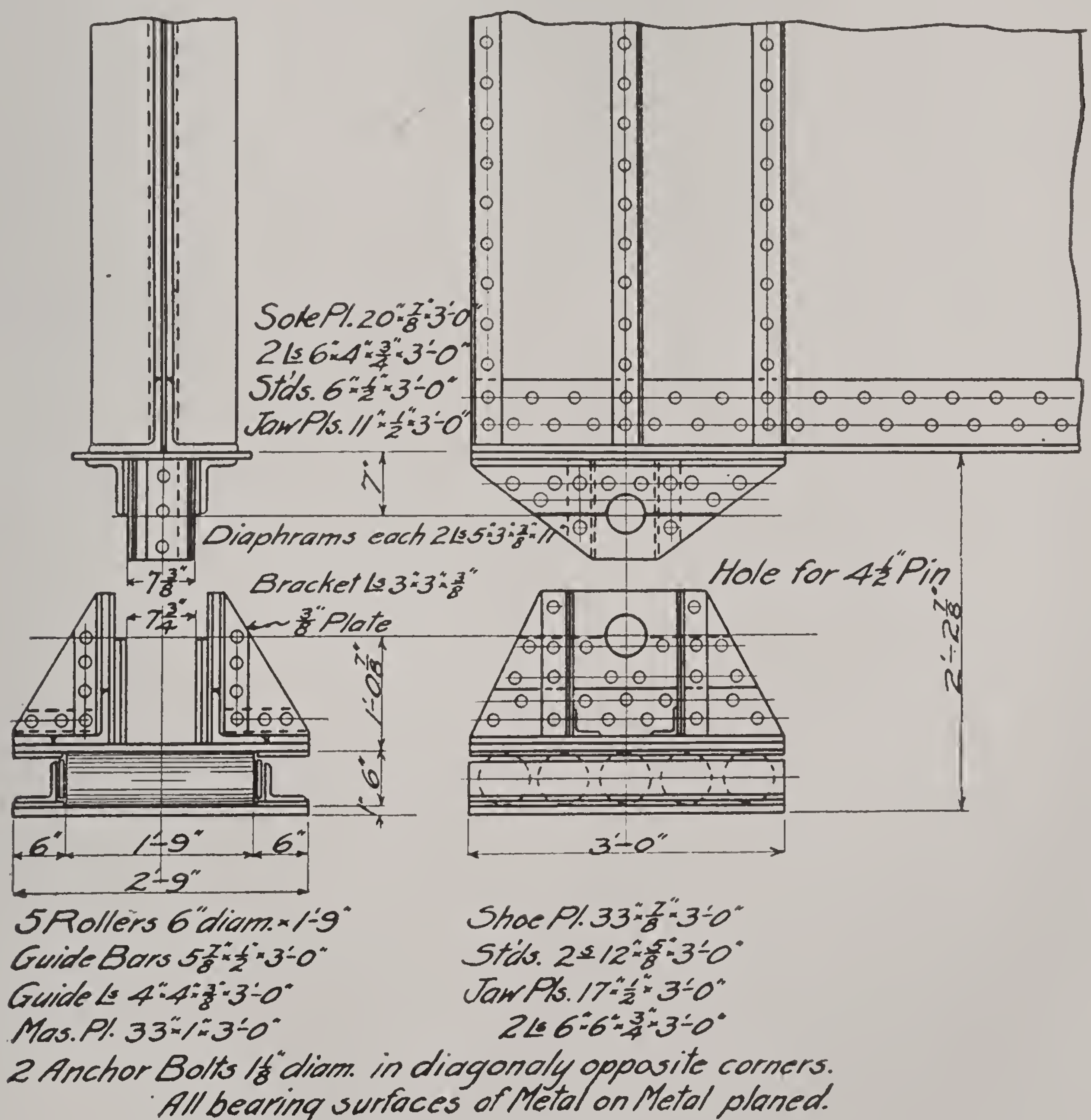


Fig. 52.

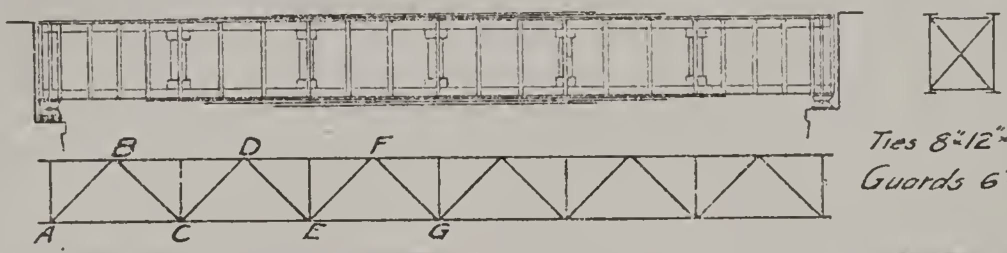
FORM No. 1

THE OHIO STATE BRIDGE COMPANY

Sheet No. 1 Made by C.T.M. Date 11-10-08

Estimate for W. P. Q. Ry. Bridge No. 217 Northern Division

Span Extreme 103'-0"	Span C. to C. 100'-0"	Estimated DL per ft. — Steel 1450	Total Steel 136700
Roadway Single Track	Panels at	Floor & Track 526	Steel per ft. 1367
Sidewalk None	Depth ~~c. to c.~~ b. to b. 10'-0½"	Total 1976	Total Lumber
Capacity Trusses E 50	Length of Diag.	Panel Load per Truss DL	11.9L+177
Capacity Floor See Spec.	Width c. to c. = 8'-0"	" " " " LL	
Specifications Am. Ry. E. & M. W.	Sec. Tg.		



Ties 8"×12"×11'-0" 14" c. to c.
Guards 6"×8"

MEM	DL Stress	LL Stress	Impact	Total Stress	Unit Stress	Req. Area	MATERIAL	Actual Area	No. Pcs.	Wt. P. Ft.	Length	WEIGHT
2 Girders							⅛ Web	5.62				
	Moments 1,235	4,025	3,018.8	8278.8		52.78	2 Ls 8"×8"×⅞"	21.23	8	45.0	103.0	37080
							1-20"×½" Top Full Bot. 75'	9.00	4	34.0	89.0	12100
	End Shear						1-20"×½"	9.00	4	34.0	60.0	8150
	494	187.5	140.6	377.5			1-20"×7/16"	7.87	4	29.8	42.0	5010
	Shear at 1st Splice						Net	52.72				
Web	32.9	130.2	102.0	265.1			1-120"×⅜"		2	153.0	103.0	31520
				End Stiff.			6 Ls 7"×3½"×7/16"		24	15.0	10.0	3600
	Shear at 2nd Spl.				Fills		= 3½"×⅞"		24	10.4	8.7	2170
	16.5	83.3	68.2	168.0								
				Int. Stiff.			2 Ls 6"×3½"×⅜" (crimp)		56	11.7	10.0	6550
	Shear Ctr.			"	"		2 Ls " " " —		20	11.7	10.0	2340
	0	46.9	40.2	87.1	Fills		1-3½"×⅜"		20	4.46	8.7	780
					"		1-3½"×⅛"		20	1.5	6.7	200
				Web Splice			2 = 13"×⅜"		20	16.6	6.7	2220
							2 = 12"×½"		40	20.4	1.6	1300
				Flange	"		1 L 8"×8"×11/16" (Cut to 7"×7")		8	35.8	4.0	1140
							1-7"×11/16"		8	16.4	6.5	850
								Rivets 4%				4600
							119	610				
2 End Frames												
				Top & Bot.			1 L 3½"×3½"×⅜"		4	8.5	7.3	250
				Diag.			1 L 5"×3½"×⅜"		4	10.4	11.8	490
				Conn. Pls.			20"×⅜"		8	25.5	2.0	410
								Rivets 4%				50
							1	200				
								Forward →				120810

FORM No.

THE OHIO STATE BRIDGE COMPANY

Sheet No. 2 Made by C.T.M. Date 11-10-08

Estimate for W.P.Q. Ry. Bridge No. 217 Northern Div.

MEM	DL Stress	LL Stress	Impact	Total Stress	Unit Stress	Req Area	MATERIAL	Actual Area	No. Pcs	Wt. P. Ft.	Length	WEIGHT
								Brought Forwd.				120810
5 Intermediate Frames												
					Top & Bot.		1L 3$\frac{1}{2}$"×3"×$\frac{3}{8}$"		10	7.8	7.3	570
					Diag.		1L " " "		10	7.8	11.8	920
					Conn. Pl.		1-18"×$\frac{3}{8}$"		20	23.0	1.7	780
								Rivets 4% →				90
							2360					
Laterals												
AB				58.5	11110	5.27	1L 6"×6"×$\frac{1}{2}$"	5.75	2	19.6	10.5	410
BC				54.1	"	4.87	1L 6"×6"×$\frac{7}{16}$"	5.06	2	17.2	10.5	360
CD				39.6	"	3.56	1L 6"×6"×$\frac{3}{8}$"	4.36	2	14.9	10.5	310
DE				36.1	9300	3.89	1L 5"×5"×$\frac{7}{16}$"	4.19	2	14.3	10.5	300
EF				24.5	"	2.64	1L 5"×5"×$\frac{3}{8}$"	3.61	2	12.3	10.5	260
FG				21.6	6710	3.23	1L 4"×4"×$\frac{7}{16}$"	3.31	2	11.3	10.5	240
					Conn. Pls.		1-24"×$\frac{3}{8}$"		2	30.6	2.5	150
							1- "		6	"	4.5	830
							1-20"×$\frac{3}{8}$"		6	25.5	4.0	610
							1-12"×$\frac{3}{8}$"		7	15.3	1.5	160
								Rivets 4% →				150
							3780					
4 Upper Shoes							1-20"×$\frac{15}{16}$" (planed to $\frac{7}{8}$")		4	63.8	3.0	770
							2Ls 6"×4"×$\frac{3}{4}$"		8	23.6	3.0	570
							2≐ 6"×$\frac{1}{2}$" (Ar.)		8	10.2	2.5	200
							2≐ 11"×$\frac{1}{2}$" (")		8	18.7	1.8	270
							4Ls 5"×3"×$\frac{3}{8}$"		16	9.8	0.9	140
								Rivets 4% →				80
							2030					
4 Lower Shoes												
							1-33"×1" (planed to $\frac{7}{8}$")		4	112.2	3.0	1350
							4≐ 12"×$\frac{5}{8}$" (Ar.)		16	25.5	2.5	1020
							2≐ 17"×$\frac{1}{2}$" (")		8	28.9	2.3	530
							2Ls 6"×6"×$\frac{3}{4}$"		8	28.7	3.0	690
							4Ls 3"×3"×$\frac{3}{8}$" (T)		16	7.2	2.3	270
							4≐ 12"×$\frac{3}{8}$" (Av.)		16	15.3	0.7	170
					Guides		2≐ 6"×$\frac{1}{2}$"		8	10.2	3.0	240
					Mas. Pl.		1-33"×1$\frac{1}{8}$" (planed to 1")		2	126.2	3.0	760
							2Ls 4"×4"×$\frac{3}{8}$"		4	9.8	3.0	120
								Rivets 4% →				200
							5350					
2 Roller Nests							5 Rounds 6"		10	96.1	1.8	1730
				Side Bars			2≐ 4$\frac{1}{2}$"×$\frac{1}{2}$"		4	7.65	3.0	90
				Stay Rods.			2 ⊙ 1"		4	2.67	3.0	30
				Tap Bolts			1" (Eq.)		20	1.0		20
							1870					
			Miscellaneous				Track bolts &c.	500	1	5	103	500
								Total Steel →				136700

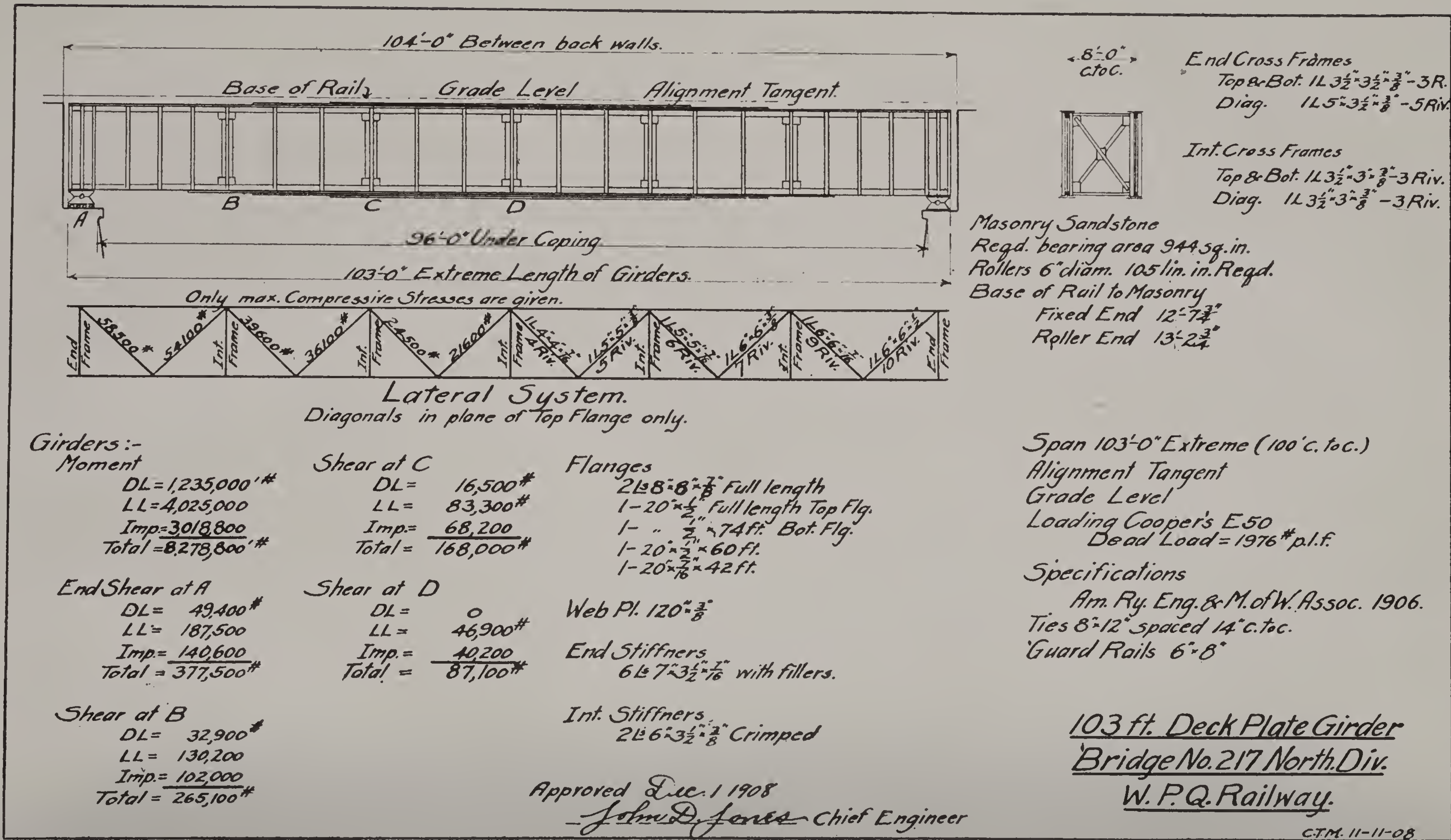

104'-0" Between back walls.
Base of Rail
Grade Level
Alignment Tangent.
A
B
C
D
96'-0" Under Coping.
103'-0" Extreme Length of Girders.
Only max. Compressive Stresses are given.
End Frame
58,500#
54,100#
Int. Frame
39,600#
36,100#
Int. Frame
24,500#
21,600#
Int. Frame
1L 4"×4"×7/16" 4 Riv.
1L 5"×5"×3/8" 5 Riv.
Int. Frame
1L 5"×5"×3/8" 6 Riv.
1L 6"×6"×1/2" 7 Riv.
Int. Frame
1L 6"×6"×1/2" 9 Riv.
1L 6"×6"×1/2" 10 Riv.
End Frame
Lateral System.
Diagonals in plane of Top Flange only.
8'-0" c. to c.
End Cross Frames
Top & Bot. 1L 3½"×3½"×3/8" - 3R.
Diag. 1L 5"×3½"×3/8" - 5 Riv.
Int. Cross Frames
Top & Bot. 1L 3½"×3"×3/8" - 3 Riv.
Diag. 1L 3½"×3"×3/8" - 3 Riv.
Masonry Sandstone
Reqd. bearing area 944 sq. in.
Rollers 6" diam. 105 lin. in. Reqd.
Base of Rail to Masonry
Fixed End 12'-7¾"
Roller End 13'-2¾"
Girders:-
Moment
DL = 1,235,000'#
LL = 4,025,000
Imp. = 3,018,800
Total = 8,278,800'#
End Shear at A
DL = 49,400#
LL = 187,500
Imp. = 140,600
Total = 377,500#
Shear at B
DL = 32,900#
LL = 130,200
Imp. = 102,000
Total = 265,100#
Shear at C
DL = 16,500#
LL = 83,300#
Imp. = 68,200
Total = 168,000#
Shear at D
DL = 0
LL = 46,900#
Imp. = 40,200
Total = 87,100#
Flanges
2Ls 8"×8"×7/8" Full length
1-20"×½" Full length Top Flg.
1- " ½" × 74 ft. Bot. Flg.
1-20"×½" × 60 ft.
1-20"×7/16" × 42 ft.
Web Pl. 120"×3/8"
End Stiffners
6Ls 7"×3½"×7/16" with fillers.
Int. Stiffners
2Ls 6"×3½"×3/8" Crimped
Approved Dec. 1 1908
John D. Jones Chief Engineer
Span 103'-0" Extreme (100' c. to c.)
Alignment Tangent
Grade Level
Loading Cooper's E50
Dead Load = 1976# p.l.f.
Specifications
Am. Ry. Eng. & M. of W. Assoc. 1906.
Ties 8"×12" spaced 14" c. to c.
Guard Rails 6"×8"
103 ft. Deck Plate Girder
Bridge No. 217 North Div.
W. P. Q. Railway.
C.T.M. 11-11-08

Fig. 53.

The difference in the actual dead load from that assumed is 1450—1367=83 lbs. per lin. foot. This would decrease the maximum moment $\frac{83\times100^2}{8}$=103,800 ft. lbs., which would decrease the required flange area at the center $\frac{103800\times12}{118.77\times16000}$=0.66 sq. in., making the total required area 52.28—0.66=51.62 sq. in. If one of the cover plates be reduced in thickness $\frac{1}{16}$ inches, the actual net area would be 51.59 sq. in., so therefore, the reduction in dead load would not allow a reduction in section of any member.

The stress sheet will now be drawn up. (See Fig. 53.)

53. Through Plate Girders. In a through plate girder bridge, all of the live and dead load, except the weight of the girders themselves, is concentrated at the panel points; and the weight of the girders may also be considered concentrated at the panel points to simplify the calculations.

The shears and moments are found, then, as for a truss bridge.[1]

The flange diagram (similar to Fig. 47) will be polygonal, the area required at each panel point being calculated.

The load on the masonry will equal the end shear of the girder plus the corresponding end reaction of the end floor beam (if an end beam is used).

There is no vertical load on the flange rivets, and the pitch of the rivets will be constant between panel points because the shear is constant.

There can be but one lateral system[2], and this is made of the Pratt type with two diagonals in each panel. It is assumed that these diagonals take tensile stresses only; they are connected to the lower flanges of the stringers so as to take up the longitudinal force due to the application of the brakes on a moving train. This tractive stress would, otherwise, produce sidewise bending in the floor beams.

The top flanges of the girders are supported at the panel points by means of solid web brackets extending down to the

[1]See Heller's "Stresses in Structures," Chapters XII and XIII.
[2]See Heller's "Stresses in Structurs," Art. 151, page 275.

top of the floor beams, and made as wide as the specified clearance will allow.

Sometimes through plate girder bridges have solid floors, in which case the moment and shear vary as for a deck plate girder.

In this case no lateral system would be necessary.

CHAPTER VI.

PIN CONNECTED BRIDGES.

54. Construction. In a truss bridge, the loads are delivered to the trusses at the panel points only. In the ordinary bridge this is done by means of floor beams and stringers. The stringers carry the ties and rails direct, and are in turn supported by the floor beams at the panel points of the truss. This construction causes the moment in the truss to vary uniformly between panel points and the shear to be uniform in each panel. (53)

55. Types of Trusses. Pin connected trusses are nearly always of the Pratt or Baltimore type, because the pin connection is not well adapted to members whose stresses alternate in direction.

The tendency is toward long panels, that is, ordinarily from 20 ft. to 25 ft., because this gives few and heavy members both in the trusses and in the floor system, and these are cheaper to manufacture and give a stiffer structure under traffic. An odd number of panels is preferable to an even number, because the maximum moment will be less and because the structure may be made symmetrical about the center line with regard to field splices.

The panel lengths must be chosen so as to give an efficient lateral system without increasing the width of the bridge beyond that required for clearance of the roadway. For a single track bridge this width is usually about 16 ft., depending, of course, on the width of the truss members.

The economic depth[1] (46) cannot be determined with any degree of certainty, but is usually taken at from one-fifth to one-sixth of the length of span. The deeper the truss is made the stiffer it will be and the less the vibratory stresses will be. It is found that considerable variation in the depth will effect the weight and cost but little. The depth must be made sufficient to allow efficient overhead bracing without interfering

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[1] See "Stresses in Bridge and Roof Trusses," by W. H. Burr, Art. 76, page 353.

with the clearances required by the traffic, unless a pony truss is used.[1]

Pin connected pony trusses are not desirable because of the lack of efficient transverse bracing.

56. Loads.[2] Most specifications give a series of wheel loads representing the weights of two locomotives, followed by a uniform train load which is to be used in designing the structures. In bridges over 100 ft. long, if an equivalent uniform load is used which will give the same center moment in the span, the errors in the stresses will not be large. It will be necessary to use several different equivalent loads for different parts of the structure, as, for instance, one for the stringers, one for the floor beams and hip verticals, and one for the trusses. The labor involved in obtaining these equivalent loadings for the floor beams and stringers is about as great as it is to calculate the stresses directly, so the wheel loads are generally used for these.

An equivalent uniform load is usually used for the trusses (except the hip verticals). The stresses in some of the members will be too large and in some too small. The variation will usually be less than about 4% from the stresses obtained by using the actual wheel loads specified.[3]

Even if the exact loading specified should ever come upon the bridge, the stresses calculated from the wheel loads would not be the true stresses, because the track distributes the wheel loads differently from what we assume, and the stringers are partially continuous while we assume them to be simply supported at each floor beam. Besides, the impact or vibratory stresses cannot be estimated with less than a probable error amounting to many times the discrepancy in the stresses obtained by the two methods.

Numerous other methods of obtaining "equivalent" loadings have been proposed, and it is probably due to the disagree-

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[1] See Heller's "Stresses in Structures," Art. 113 and 151.

[2] See Heller's "Stresses in Structures," Art. 119, 130 and 131.

[3] See Trans. Am. Soc. C. E., Vol. 42, pp. 189, 215 and 206.
Also Johnson's "Modern Framed Structures," Chapter VI.
Also DuBois' "Stresses in Frames Structures."

ment on this point that engine loads are still specified in all but a few specifications, and that some engineers still calculate all stresses from wheel loads.

The *dead load* must be estimated. The weight of the floor (52) including rails, ties and guards for the ordinary floor construction, with stringers not over 6 ft. 6 in. center to center, will not exceed about 400 lbs. per linear foot of track. The weight of the steel work may be estimated approximately by comparison with some previously made estimates of similar structures, or may be taken from an empirical formula (52). After the design and estimate are completed, the dead load must be revised to agree with the final estimated weight, if the discrepancy is sufficient to change the size of any of the members. (51) (52).

57. Tension Members. The tension members of pin connected trusses, except the hip verticals, and in some cases the counters and end panels of the lower chord, are usually made of eye bars. The counters and end panels of lower chord are frequently required to be made rigid members, to increase the stiffness of the bridge. The hip verticals should always be rigid members, because this gives a better connection for the floor beams at these points, and because it greatly reducs the vibration.

Eye bars are forged, and the heads are made of such size that in testing, the bar will break in the body instead of through the head. Usually the net section through the pin hole is made about 25% in excess of the section through the body of the bar.[1]

Eye bars should not be thinner than about ¾ inch, and should not be too thick, say over about 2¼ inches. Thick bars will usually not show as high an ultimate strength as thin ones. The usual proportions of width to thickness lie between 3 to 1 and 7 to 1.

Built tension members, of course, contain more material than tension members of the same strength made of eye bars. The net section through rivet holes (11) and through the pin holes must be carefully investigated. The most common form

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[1] Sizes of eye bars as manufactured by that company are given in Cambria, page 333.

is an I cross section made of four angles latticed together, although two channels latticed are frequently used. Sometimes two eye bars are laced together with bent bars, but this does not give a member much stiffer than the plain eyebar member.

The required net area of a tension member is obtained by simply dividing the stress in the member by the allowed unit stress in tension as given by the specifications.

58. Compression Members. The intermediate posts are usually made of two channels, either built or rolled, latticed together. Built channels are of course more expensive than rolled channels, on account of the extra punching and riveting.

If the toes of the channels are turned in, the backs form plane surfaces to which connections may be more easily made than if the toes are turned out. The distance in the clear between the channels must be great enough to allow the entrance of the riveting tool between the lacing bars, and it is economical to place the channels far enough apart to make the ratio of the unsupported length to the radius of gyration in the two directions equal.[1] Local conditions frequently limit the dimensions of these members.

Experiments show that a column will fail at an average unit stress over the entire cross section, which is less than the ultimate strength of the material in compression, and that the longer the column the less will be this average unit stress at failure. In other words, a column does not fail by compression alone but by a combination of compression and bending.[2]

This is taken into account in the design of compression members by the use of a *"column formula,"* which gives us an average unit stress which it is safe to allow on the cross section, and after this unit is determined the design of the compression member is as simple as that of a tension member, but the determination of the average unit stress allowable, involves properties of the cross section of the member, so the solution must be by trial.

[1] See Cambria, page 221.

[2] For an excellent article on Columns see editorial in Engineering News, January 3, 1907.

A column formula consists of a variable reduction factor applied to the maximum allowed fiber stress in compression.

The column theory[1] assumes that the whole member acts as one piece, and the function of the stay plates and lacing is to hold the component parts of the member in line and to insure its action as a unit.

A column under stress will deform into a curve with a point of contra-flexure near each end,[2] the distance from the end depending upon the degree of fixity of the end. At these points of contra-flexure the bending moment is zero, and consequently the stress on the column cross section is uniform. Midway between these points the maximum bending moment occurs, and the maximum unit stress in compression occurs on the concave side, therefore in a distance equal to one half the length between the points of contra-flexure, the unit stress in the concave side of the column must change from the average to the maximum allowed.

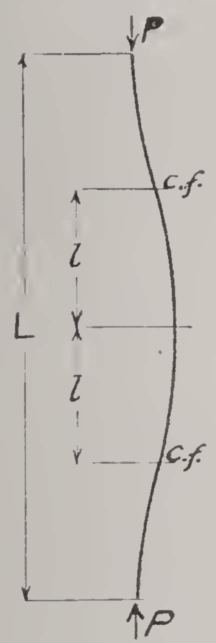

Fig. 54.

Suppose a column to be made up of two leaves connected by lacing or otherwise.

Let s_1=maximum allowed unit stress on the material in compression.

s_c=average unit stress over the cross section$=\frac{P}{A}$

F=total change in stress in one leaf of the column in a distance l.

f=change in the total stress in one leaf per unit of length$=\frac{F}{l}$

l=the least distance from the point of maximum bending moment to a point of contra-flexure.

L=total length of column.

A_1=area of cross section of one leaf.

$$F=A_1\,(s_1-s_c) \qquad (16)$$

$$f=\frac{A_1\,(s_1-s_c)}{l} \qquad (17)$$

[1] See Seller's "Stresses in Structures," Chapter X.
[2] See Heller's "Stresses in Structures," Fig. 137, page 178.

For a pin ended column $L=2l$ and for a square or fixed ended column $L=4l$. Any column in practice will lie somewhere between these two limits, and in any case eccentricities of manufacture and loading may make l different than theory would indicate.

Also this theory assumes that the rate of change of stress in the leaf is uniform, which is not true, therefore, to be on the safe side we will take $L=4l$ in all cases; then

$$f=\frac{4A_1(s_1-s_c)}{L} \tag{18}$$

Equation (18) gives the longitudinal increment of stress in one leaf per unit of length of column, and sufficient connection must be provided between the leaves to transmit this stress (49). The values of s_1 and s_c are taken from the column formula which is being used unless there is bending due to transverse loads. (77)

When lacing[1] is used the bars must be capable of taking their stress either in tension or compression.

The top chords and end posts are usually made of two built or rolled channels, connected by a cover plate on top and by stay plates and lacing on the bottom. The cover plate being solid aids in taking compression, and its area is always considered in the effective cross section. The cover plate then serves both as a part of the compression area and to tie the two leaves of the column together.

A compression member with a cover plate on one side only is not symmetrical about its center of gravity, and the end connections must be designed to transmit the stresses to the cross section properly. (10)

The cover plates should always be made as thin as the specifications will allow unless they have some special duty to perform, so as to keep the eccentricity of the section small. The unsupported width of plates in compression (distance between rivet heads) is usually limited by the specifications to 30 or 40 times the thickness of the metal.

If a compression member is subjected to transverse loads,

[1] For various methods of calculating lacing see Report of the Royal Commission on the failure of the Quebec Bridge, Appendix No. 16. Also the Report of C. C. Schneider.

causing bending[1] in addition to the direct load (40), the maximum fiber stress due to both *must* not exceed the maximum allowed unit compressive stress (s_1), and to be on the side of safety *should* not exceed a unit stress determined by a suitable column formula (s_c), because the accidental eccentricities *may increase* the bending due to the transverse loading.

The horizontal and inclined compression members are in bending due to their own weight in addition to being in compression. In the top chords and end posts of bridges this bending moment is partially neutralized by lowering the centers of the end connections an amount sufficient to produce an upward bending moment due to the eccentricity of the compressive stress, equal to the downward bending moment due to weight.

$$Pe = \frac{wL^2}{8} \quad \text{and} \quad e = \frac{wL^2}{8P} \tag{19}$$

Equation (19) is generally used in practice to determine the eccentricity of the pins to compensate for the bending due to the weight of the member. Using this value of e would render the bending moment almost zero at the middle, but as the bending moment (Pe) due to the eccentricity is a constant, while the moment due to the weight is a maximum at the middle and zero at the ends, the use of this value of e produces a negative bending moment at the end as great as the original moment due to the weight.[1] It is better to use a smaller value of e as given by equation (20),

$$e = \frac{wL^2}{10P} \tag{20}$$

as this will give a less resultant maximum bending moment in the column.

Another case in which compression members are subjected to both axial and bending stresses is the end posts of a through bridge with overhead bracing. The end posts must carry the wind load in bending from the portal to the abutments.[2] This bending is in a plane perpendicular to that of the bending due to weight. The lacing and riveting of the cover plates of the

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[1] See Heller's "Stresses in Structures," Art. 111, page 190.

[1] See Article by Prof. J. E. Boyd in Engineering News for April 11, 1907, page 404.

[2] See Heller's "Stresses in Structures," Arts. 153 to 165 inclusive.

end posts must be sufficient to transfer the increments of stress as determined by equation (17).

59. Lateral Systems.[1] In a through bridge a lateral system is always provided in the plane of the lower chord and, if the head room permits, in the plane of the upper chord also. In a deck truss bridge, lateral systems should always be provided in the plane of both the top and bottom chords.

The top lateral system in a deck bridge and the bottom lateral system in a through bridge is assumed to take all the wind load on half the projection of the trusses, the floor system and the train and the centrifugal force if the track is on a curve,[2] although a small part of the latter would be transmitted to the top lateral system by the stiffness of the intermediate posts.

The lateral system is a horizontal Pratt truss in which the floor beams act as the posts and the chords of the main trusses act as chords. The diagonal members are put in in both directions to provide for a reversal of wind.

The top lateral system in a through bridge and the bottom lateral system in a deck bridge take the wind load on half the projection of the trusses.

The end reaction of the top lateral system in a through bridge is conveyed to the abutment by means of *portal bracing* between the end posts and by bending in these end posts.[3] (58)

Provision must be made in all main truss members carrying wind stresses for these, in addition to the dead and live load stresses.

The wind blowing upon the side of a train on a bridge tends to overturn it, and thus produces a greater load on the leeward truss than on the windward. The effect on the top chord of a through bridge is very small because the leeward top chord would be in tension under the wind load alone. The bottom chords and web members should, however, be proportioned for this additional stress. (63)

60. Design of a Pin-connected Railway Bridge. To illustrate the methods of solving the various problems connected

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[1] See Heller's "Stresses in Structures," Chapter XIV.
[2] See Heller's "Stresses in Structures," Art. 166.
[3] See Heller's "Stresses in Structures," Arts. 153 to 165 inclusive.

with the design of truss bridges, the design for a through Pratt truss railway bridge will now be worked out.

We will assume the following data:

Span 189 ft. c. to c. of end pins = 7 Panels at 27 ft.
Single track. Alignment tangent.
Specifications Cooper's 1906 for Railway Bridges.
Loading Cooper's E 40.
Material medium steel except rivets.

61. Dead Load. (56) The stringers will be spaced 6 ft. 6 in. c. to c., and the size of the tie may be calculated as was done in Art. 52. We will use 8″x 8″ ties 10 ft. long spaced 14 in. c. to c., guard rails 6″x 8″. The weight of the floor comes out somewhat less than 400 lbs. per lin. ft., but the specification §23 directs that not less than 400 lbs. per linear ft. shall be used. (51)

The weight of the steel work may be approximately estimated from equation (15), $w=7L+600$ from which $w=1923$ lbs. per lin. ft. of bridge. The total dead load then will be $1923+400=2323$ lbs. per lin. ft. of bridge, one-third of which will be considered as acting at the top chord and two thirds at the bottom chord.

62. The Depth of the trusses (55) must be sufficient to allow the required head room and also an efficient portal. The depth of the floor system will govern this to some extent also.

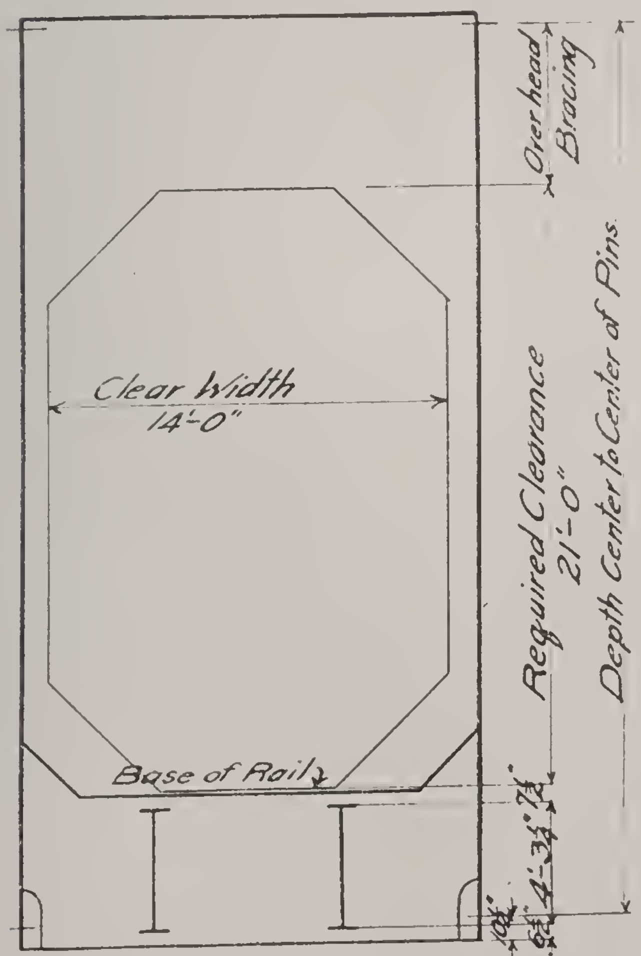


Fig. 55.

An estimate of the depths required for these various parts may be made and an approximate minimum allowed depth calculated in the form of a table similar to the one outlined in Art. 28.

The stringer may be designed before this table is made up, as the depth of the truss does not effect it. We will use the stringer as designed in Art. 51, for this bridge.

Depth of tie over stringer. . . . . $0'$-$7\frac{1}{2}''$ (Spec. §12)
Depth of stringer . . . . $4'$-$3\frac{1}{4}''$
Bot. of stringer to bot. of Fl. Bm. . . . . . $0'$-$6\frac{1}{2}''$

$5'$-$5\frac{1}{4}''$

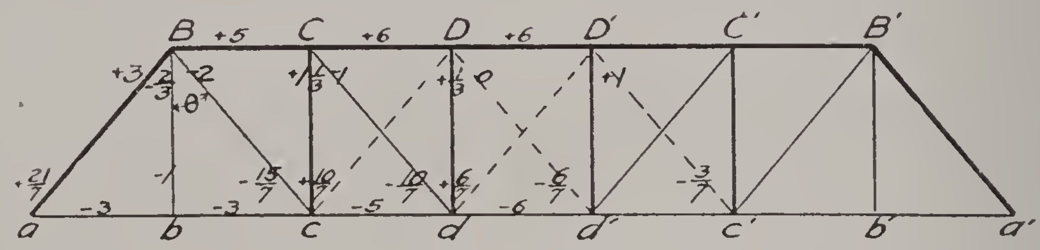
Bot. of Fl. Bm. to Pin Cent. . . . . $10\frac{1}{4}$

Base of Rail to Pin Cent. . . . . $4'$-$7''$
Required Clearance . . . . $21'$-$0''$ (Spec. §4)
Portal depth say . . . . $4'$-$5''$ (min.)

Total depth c. to c. of pins. . . . . $30'$-$0''$ (min.)

The depth should be about $\frac{1}{5}$ to $\frac{1}{6}$ the span (55), so we will use a depth of $32'$-$0''$ c. to c. of pins.

63. Stresses. For all of the truss members except the hip verticals, an equivalent uniform live load will be used,



Sec. $\theta = \frac{41.87}{32}$ Fig. 56. Tan. $\theta = \frac{27}{32}$

Mem.	Dead Load Stresses	Live Load Stresses: Equiv. Uniform	Live Load Stresses: Wheel Loads	Wind Stresses: From Lat. Sys.	Wind Stresses: From Overturning	Wind Stresses: Total Max.	Remarks
aB	+123100	+255400	+264500		∓ 31300	+ 31300	See Portal
Bc	− 82100	−182400	−190000		± 22400	− 22400	Neglect Wind
Cd	− 41000	−121600	−126700		± 14900	− 14900	" "
Dd′	− 0	− 73000	− 74000		± 9000	− 9000	" "
D′c′	+ 41000	− 36500	− 34500		± 4500	− 4500	" "
C′b′	+ 82100	− 12100	− 9600		± 1500	− 1500	No Member
Bb	− 20900	−	− 80100		± 8000	− 8000	Neglect Wind
Cc	+ 41800	+ 93000	+ 96900		∓ 11400	+ 11400	" "
Dd	+ 10500	+ 55800	+ 56600		∓ 6900	+ 6900	" "
ab	− 79400	−164700	−170600	+81600 −0	± 20200	+101800 −20200	" "
bc	− 79400	−164700	−170600	+136000 −81600	± 20200	+156200 −101800	Wind exceeds 30%
cd	−132300	−274500	−274500	+163200 −136000	± 33700	−169700	" " "
dd	−158800	−329500	−330100	± 163200	± 40400	−203600	" " "
BC	+132300	+274500	+274500	+18100 −0	∓ 33700	+ 33700	Neglect Wind
CD	+158800	+329500	+330100	+27200 −18100	∓ 40400	+ 27200	" "
DD′	+158800	+329500	+330100	± 27200	∓ 40400	+ 27200	" "

derived from the maximum live load moment for the span. (56) This equivalent uniform load for a 189 ft. span is 4820 lbs. per lin. ft. of track.

$$\text{Panel load of Dead Load} = \frac{2323 \times 27}{2} = 31{,}360 \text{ lbs.}$$

$$\text{Panel load of Live Load} = \frac{4820 \times 27}{2} = 65{,}070 \text{ lbs.}$$

The table under Fig. 56 gives the direct stresses in all of the truss members due to dead load, live load, and wind. The live load stresses calculated from the wheel loads are also given in a parallel column for comparison. The maximum error is seen to be in the end posts and amounts to about 3½%.

The wind stresses in the chords from the lateral systems are gotten by assuming that the trusses are 16′-0″ c. to c. This will not be far from right.

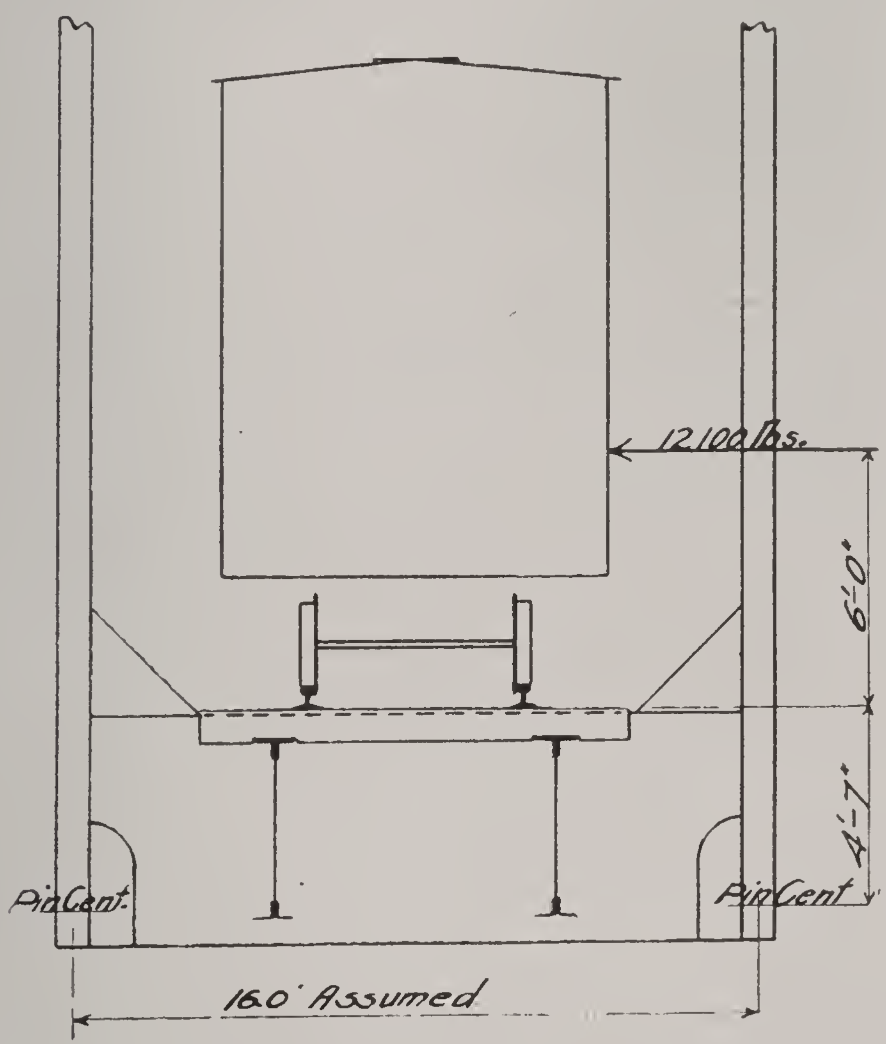


Fig. 57.

According to specifications, §24, 450 lbs. of the wind load shall be treated as acting on a moving train at a height of six feet above the base of rail. This gives a height of $6.0 + 4.58 = 10.58$ feet above the pin centers. The horizontal force acting at this height (see Fig. 57) will be $450 \times 27 = 12{,}100$ lbs. per panel. The additional load on the leeward truss at each panel due to this overturning moment will be

$$V = \frac{12100 \times 10.58}{16} = 8{,}000 \text{ lbs.}$$

The additional stress in each member then will be the direct live load stress in the member, (figured for the equivalent uniform load) multiplied by $\frac{8000}{65070}$

The specifications §39 directs that the stresses in the truss members due to wind may be neglected unless they exceed 30% of the combined dead and live load stresses. Therefore we will have to consider the wind stresses only in the bottom chords. The bending in the end posts due to the portal stresses will be taken up in Art. 66.

64. Design of Tension Members. The required net area for any tension member is obtained by adding algebraically, the areas required for dead load and live load stresses. (Spec. §31 and 35). There is also a limiting clause for counters. (Spec. §50 and 51.)

Since in these specifictiations, the dead load unit stress is just twice the live load unit, the same area will be obtained if the live load stress plus *half* the dead load stress be divided by the live load unit stress.

From this relation we may derive an average unit stress which may be applied to the total dead plus live load stress in any member as follows:

Let s_W=this average unit stress.

s_D=the dead load unit stress.

s_L=the live load unit stress.

D=the dead load stress in the member.

L=the live load stress in the member.

Then

$$\frac{\frac{1}{2}D+L}{s_L}=\frac{D+L}{s_W} \qquad s_W=\frac{2(D+L)s_L}{D+2L}=\frac{2s_L}{1+\frac{L}{D+L}}$$

$$s_D=2s_L \quad s_W=\frac{s_D}{1+\frac{L}{D+L}} \tag{21}$$

It is not necessary to find this average unit stress except for those members in which the wind stress must be taken into account according to specifications §39.

The simplest members, those made up of eye bars, will be proportioned first. We will assume that we are limited in the choice of eye bars to those manufactured by the Cambria Company, as indicated in their hand book, pages 332 and 333.

Bc Required D. L. Area$=\dfrac{82100}{20000}=4.11$ sq. in.

Required L. L. Area$=\dfrac{182400}{10000}=18.24$ sq. in.

Total$=22.35$ sq. in.

This may be made up of 4 bars $6''\text{x}1\frac{5}{16}''$ (area=22.50 sq. in.) or 2 bars $7''\text{x}1\frac{5}{8}''$ (area=22.76 sq. in.). The 6 inch bars are slightly more economical and will not require such large heads so we will use 4 bars $6''\text{x}1\frac{5}{16}''$ for *Bc*.

Cd Required D. L. Area$=\dfrac{41000}{20000}=2.05$ sq. in.

Required L. L. Area$=\dfrac{121600}{10000}=12.16$ sq. in.

Total$=14.21$ sq. in.

This will require 2 bars $6''\text{x}1\frac{3}{16}''$ (area=14.25 sq. in.)

Dd′ Required D. L. Area=00

Required L. L. Area$=\dfrac{73000}{10000}=7.3$ sq. in.

This will require 2 bars $4''\text{x}\frac{15}{16}''$ (area=7.50 sq. in.) or 2 bars $3''\text{x}1\frac{1}{4}''$ (area=7.50 sq. in.). The 3 inch bars cannot be used because, probably the size of the pin at *d* will exceed 5 in., which is the largest size that the table in Cambria gives for a 3 inch bar, so we will use 2 bars $4''\text{x}\frac{15}{16}''$

An increase in live load of 25% or to E50, will increase the unit stress in this counter exactly 25% so §51 of the specifications is satisfied.

D′c′ Required D. L. Area$=\dfrac{41000}{20000}=2.05$ sq. in.

Required L. L. Area$=\dfrac{36500}{10000}=3.65$ sq. in.

Difference$=1.60$ sq. in.

To comply with specifications §51 an increase in live load of 25% must not increase the unit stresses more than 25% therefore:

Required D. L. Area$=\dfrac{41000}{20000+25\%}=1.64$ sq. in.

Required L. L. Area$=\dfrac{36500+25\%}{10000+25\%}=3.65$ sq. in.

Difference$=2.01$ sq. in.

This will require 1 bar $1\frac{7}{16}$ in. square (area=2.07 sq. in.).

cd. The average unit stress allowable, as determined by equation (21) must be used here because the wind stress is more than 30% of the dead and live load stresses. (Spec. §39.)

$$s_w = \frac{20000}{1+\dfrac{274.5}{406.8}} = 11{,}940 \text{ lbs. per sq. in.}$$

$$11{,}940+30\% = 15{,}520 \text{ lbs. per sq. in.}$$

Total stress in cd=132.3+274.5+169.7=576.5.

$$\text{Required Area} = \frac{576500}{15520} = 37.14 \text{ sq. in.}$$

This will require 4 bars $6''\text{x}1\frac{9}{16}''$ (area=37.50 sq. in.) or 2 bars $7''\text{x}1\frac{5}{16}''$ plus 2 bars $7''\text{x}1\frac{3}{8}''$ (area=37.64 sq. in.)

The 7 inch bars will be better because the thickness is less, and this will give a less bending moment on the pin, and also probably the next chord dd' will necessarily be made of 7 inch bars, in which case the same dies may be used for making all of the eye bar heads in the bottom chords, which would reduce the cost.

dd' Total stress=158.8+329.5+203.6=691.9
Allowed unit stress=15,520 lbs. per sq. in.

$$\text{Required Area} = \frac{691900}{15520} = 44.58 \text{ sq. in.}$$

This will require 2 bars $7''\text{x}1\frac{5}{8}''$ plus 2 bars $7''\text{x}1\frac{9}{16}''$ (area =44.64 sq. in.).

According to the specifications §10 the vertical suspenders and the two end panels of lower chord must be made rigid members.

abc Total stress=79.4+164.7+101.8=345.9
Allowed unit stress=15,520 lbs. per sq. in.

$$\text{Required Area} = \frac{345900}{15520} = 22.29 \text{ sq. in.}$$

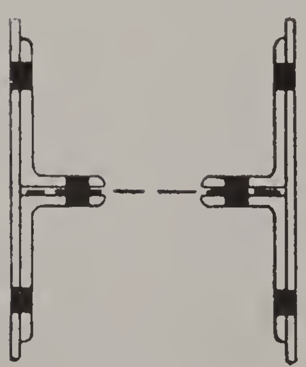


Fig. 58.

This member may be made up of 4 angles and 2 plates laced together horizontally as shown in Fig. 58. We will use 4 angles $6''\text{x}3\frac{1}{2}''\text{x}\frac{1}{2}''$, and 2 plates 14 inches wide by as thick as may be necessary to make up the required net area. To comply with specifications §64 at least two rivet holes must be deducted from each angle. (11)

Net area 4 angles 6″x3½″x½″ = 18.00 — 8x1x½ = 14.00 sq. in.

Required net area of plates=22.29—14.00=8.29 sq. in.
Equivalent net width of plates=14—2x1=12 inches.

Required thickness of plates$=\frac{8.29}{12}=$0.69 inches.

Use 2 plates 14″x⅜″. Total actual net area

2 Angles	6″x3½″x½″=	14.00 sq. in. net
2 Plates	14″x⅜″=	9.00 sq. in. net
	Total=	23.00 sq. in. net

It would be more economical to use four angles without cover plates, but it is not well to use metal thicker than about ⅝ inch in a riveted tension member, besides, according to specifications §129, material over ⅝ inch in thickness in tension members must be reamed, which would increase the cost considerably.

Bb will be made of two rolled channels, and will be made the same width as the intermediate posts *Cc* and *Dd* so that the floor beams may all be made alike.

The allowed unit stresses are less for verticals carrying floor beams than for other truss members (Spec. §31).

$$\text{Required D. L. Area}=\frac{20900}{16000}=1.31 \text{ sq. in.}$$

$$\text{Required L. L. Area}=\frac{80100}{8000}=10.01 \text{ sq. in.}$$

Total=11.32 sq. in.

There will be pin plates on the webs of the channels for the connection at *B*, and stay plates riveted to the flanges near them. These will make it necessary to take at least 4 holes out of each channel as shown in Fig. 59.

The member cannot be made of less than 10 inch channels or there would not be room for the floor beam connection. The lightest weight 10 inch channel cannot be used because specifications §82 requires that no metal less than ⅜ inches thick be used, and the web of a 10″ by 15 lb. channel is only 0.24″ thick.

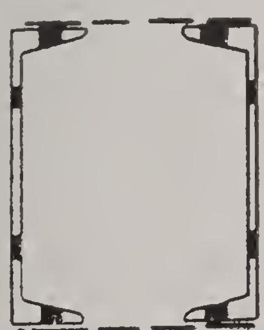


Fig. 59.

2—10″×20 lb. channels is about the smallest section that can be used.

The rivets in the flanges cannot be larger than ¾″, (see *Cambria*, page 53), while it is desirable to have the rivets in the web ⅞″ for the floor beam connections.

The net area then of the 2 channels 10″x20 lb.=11.76—4x$\frac{7}{16}$x⅞—4x0.38x1=8.71 sq. in. As this is less than the required area we must use heavier channels. Try 2 channels 12″x25 lbs. Net area=14.70—4x½x⅞—4x0.39x1=11.39 sq. in. These will answer.

65. Design of Compression Members. The least allowable section for a post is 2 channels 10″x20 lbs. (See Spec. §§35 and 82.) The greatest allowed length for a post composed of these channels is 100 times the radius of gyration (Spec. §35)=100×3.66=366 inches=30′—6″, which is less than the depth of our truss, so a larger section must be used. Try 2 channels 12″x25 lbs. Allowed length=100×4.43=443″=36′—11″.

$$\text{Allowed unit stress for D. L.}=17000-90\frac{L}{r}$$

$$=17000-\frac{90\times32\times12}{4.43}=9200 \text{ lbs. per sq. in.}$$

Allowed unit stress for L. L.= 4600 lbs. per sq. in.

Dd Required D. L. Area$=\frac{10500}{9200}=1.14$ sq. in.

Required L. L. Area$=\frac{55800}{4600}=12.13$ sq. in.

Total=13.27 sq. in.

Actual area=2×7.35=14.70 sq. in. which is sufficient.

Cc. The stresses for this post are considerably greater than for *Dd*, so we will try 2 channels 15″x33 lbs.

Allowed D. L. unit stress=10,850 lbs. per sq. in.
Allowed L. L. unit stress= 5,420 lbs. per sq. in.

Required D. L. Area$=\frac{41800}{10850}=3.86$ sq. in.

Required L. L. Area$=\frac{93000}{52400}=17.16$ sq. in.

Total=21.02 sq. in.

Actual area 2 channels 15″x33 lbs.=2×9.90=19.80 sq. in. Try 2 channels 15″x40 lbs.

Allowed D. L. unit stress=10,650 lbs. per sq. in.
Allowed L. L. unit stress= 5,320 lbs. per sq. in.

Required D. L. Area$=\frac{41800}{10650}=3.93$ sq. in.

Required L. L. Area$=\frac{93000}{5320}=17.48$ sq. in.

Total=21.41 sq. in.

Actual area 2 channels 15″x40 lbs.$=2\times11.76=23.52$ sq. in., which is sufficient.

In the above calculations for the posts we have assumed that the ratio of unsupported length to radius of gyration in the direction parallel to the webs of the channels was greater than that in the other direction (58). In order to make the posts safe according to the specifications, the distance between channels must be sufficient so that the allowed unit stress is greater than the actual.

The actual unit stress on $Cc=\frac{134800}{23.52}=5730$ lbs. per sq. in.

From equation (21) the allowed unit stress

$$=\left[17000-90\frac{L}{r}\right]\left(\frac{1}{1+\frac{93}{134.8}}\right)=10060-53.25\frac{L}{r}$$

Equating these two units we get

$$5730=10060-53.25\,\frac{21\times12}{r} \text{ from which } r=3.1 \text{ in.}$$

This is assuming that the post is supported by sway bracing at an elevation of 21 ft. above the floor beam. (Spec. §107).

The distance apart of the channels necessary to make the radius of gyration equal to 3.1 in. will now be calculated.

$$r=\sqrt{\frac{I}{A}} \qquad I=I_o+Ad^2 \text{ in which}$$

I_o=moment of inertia of channels about their own center of gravity.

d=distance from the center of gravity of the column to the center of gravity of the channel.

$$r=\sqrt{\frac{2\times9.39+2\times11.76\times d^2}{2\times11.76}}=3.1 \text{ from which}$$

$$d^2=8.81 \qquad d=2.97 \text{ inches}$$

The minimum distance back to back of channels (toes turned

in) then will be $2\times2.97+2\times0.78=7.5$ inches, but Spec. §35 says that the least width of post permissable is 10 inches, so we will use this width.

The post Dd will be made the same distance back to back in order that the floor beams may be made alike.

The width of the *top chord and end posts* must be made the same throughout, and the depth of the two are usually made the same, although this is not absolutely necessary.

The width must be sufficient to allow all of the web members to connect to the pins inside of the top chord section. In some cases a pair of eye bars are allowed to connect to the pin outside the chord section, but this is unusual.

There are more members connecting at B than at any other point, so the required width there will be determined.

We can only estimate this width approximately at present.

Width of Bb back to back or channels	$= 10''$
Pin plates on Bb say $2-\frac{1}{2}''$	$= 1''$
Bars Bc $2-\frac{15}{16}''$ (2 inside Bb)	$=1\frac{7}{8}''$
Webs of end posts say $2-\frac{3}{4}''$	$=1\frac{1}{2}''$
Pin plates on end posts say $2-\frac{1}{2}''$	$= 1''$
Top angles on end posts say $2-3''$	$= 6''$
Clearances say	$1\frac{1}{8}''$
	Total$=22\frac{1}{2}''$

In order to make sure that we have sufficient clearance we will make the cover plate of the chord 24 inches wide.

The depth of the chord section must be made sufficient so that there is room between the pin and the cover plate for the connection of the members. The pin will be somewhat above the geometric center of the web plates because the center of gravity of the section will be above the center of the web.

Assuming this eccentricity of the pin to be $1\frac{1}{2}''$ and that the radius of the largest eye bar head is not over $7\frac{1}{2}''$, we have $9''$ as the half depth of the web required by the clearances.

Figure 60 shows about the dimensions of the minimum chord section which we can use here.

The specification §80 requires that the thickness of plates in compression shall not be less than $\frac{1}{30}$ of the unsupported width, except for the cover plates of top chords and end posts which are limited to $\frac{1}{40}$ The unsupported width of the plate is the distance between rivet heads. For the cover plate it will be (1) $21\frac{1}{4}''-1\frac{7}{16}''=19\frac{13}{16}''$. The minimum allowed thickness of cover plate then will be $\frac{19.8}{40}=\frac{1}{2}''$. The minimum allowed thickness of web plate will be $\frac{13.3}{30}=0.45''$ or say $\frac{1}{2}''$ also.

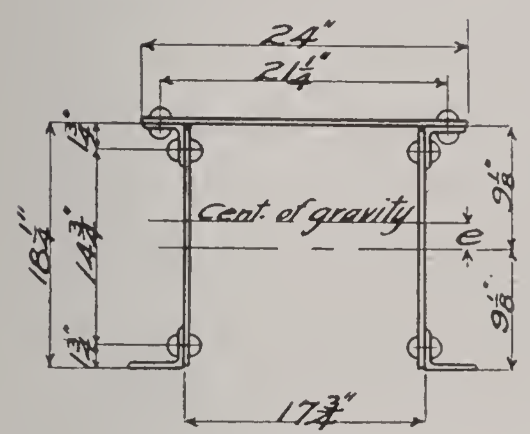


Fig. 60.

For the chord sections CD and DD we will try the following, which is about the least chord section allowable as we have seen:

2 Web plates	$18''\times\frac{1}{2}''=$	18.00 sq. in.
1 Cover plate	$24''\times\frac{1}{2}''=$	12.00 sq. in.
2 Top angles	$3''\times3''\times\frac{3}{8}''=$	4.22 sq. in.
2 Bot. angles	$4''\times3''\times\frac{3}{8}''=$	4.98 sq. in.
	Total=	39.20 sq. in.

We will now find the location of the center of gravity of the cross section by taking moments of areas about the upper side of the cover plate (see Fig. 60).

Cover plate	$12\times$	$0.25=$	3.00
Top angles	$4.22\times$	$1.39=$	5.86
Web Plates	$18.00\times$	$9.62=$	173.25
Bot. Angles	$4.98\times$	$17.97=$	89.49
		Sum=	271.60

Distance of center of gravity from top of cover plate $=\frac{271.60}{39.20}=6.93$ inches.

The distance of the center of gravity above the center of the web plate$=9.63-6.93=2.7$ inches.

The least radius of gyration is required to be used in the determination of the allowed unit stresses. This will be about a horizontal axis through the center of gravity, and is equal to $\sqrt{\frac{I}{A}}$

The moment of inertia about this axis must now be found.[1] This is done by adding to the moment of inertia of each constituent part the product of its area by the distance squared, of its center of gravity from the center of gravity of the section.

$$\begin{array}{lr}
\text{Cover plate } \dfrac{24\times(\frac{1}{2})^3}{12} = & 0.25 \\
12.00\times(6.68)^2 = & 535.47 \\
\text{Top angles } \quad 2\times1.76 = & 3.52 \\
4.22\times(5.54)^2 = & 129.52 \\
\text{Web plates } \dfrac{2\times\frac{1}{2}\times(18)^3}{12} = & 486.00 \\
18.00\times(2.70)^2 = & 131.22 \\
\text{Bot. angles } \quad 2\times1.92 = & 3.84 \\
4.98\times(11.04)^2 = & 606.97 \\
\hline
\text{Total Moment of Inertia} = & 1896.79
\end{array}$$

From which we get the radius of gyration about the horizontal axis through the center of gravity

$$r=\sqrt{\frac{I}{A}}=\sqrt{\frac{1896.79}{39.20}}=6.95 \text{ inches.}$$

The allowed dead load unit stress is $20000-90\dfrac{L}{r}=15{,}800$ from specifications §35.

$$\text{Required D. L. Area}=\frac{158800}{15800}=10.05 \text{ sq. in.}$$

$$\text{Required L. L. Area}=\frac{329500}{7900}=41.71 \text{ sq. in.}$$

$$\text{Total}=51.76 \text{ sq. in.}$$

The area of cross section must therefore be increased. We will try the following:

2 Web plates	18″×¾″	=27.00 sq. in.
1 Cover plate	24″×½″	=12.00 sq. in.
2 Top angles	3″×3″×$\frac{7}{16}$″	= 4.88 sq. in.
2 Bot. angles	4″×3″×⅝″	= 7.98 sq. in.
	Total	=51.86 sq. in.

Eccentricity=1.68 in. $\quad I=2494 \quad r=6.93$ in.

The change in the radius of gyration is seen to be very small,

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[1] See Heller's "Stresses in Structures," Art. 67, page 92.

and the corresponding change in the allowed unit will be very small, so this section will answer.

BC. The section for this chord will lie somewhere between the two tried above, and therefore we may use the same allowed unit stresses.

$$\text{Required D. L. Area} = \frac{132300}{15800} = 8.38 \text{ sq. in.}$$

$$\text{Required L. L. Area} = \frac{274500}{7900} = 34.75 \text{ sq. in.}$$

$$\text{Total} = 43.13 \text{ sq. in.}$$

Use the following section

2 Web plates	$18'' \times \frac{9}{16}''$	$= 20.25$ sq. in.
1 Cover plate	$24'' \times \frac{1}{2}''$	$= 12.00$ sq. in.
2 Top angles	$3'' \times 3'' \times \frac{3}{8}''$	$= 4.22$ sq. in.
2 Bottom angles	$4'' \times 3'' \times \frac{9}{16}''$	$= 7.26$ sq. in.
	Total	$= 43.73$ sq. in.

Eccentricity $= 1.98$ in. $\quad I = 2230 \quad r = 7.14$ in.

66. Design of the End Posts. Before the *end posts* can be designed the stresses in them due to the *portal bracing* must be determined.

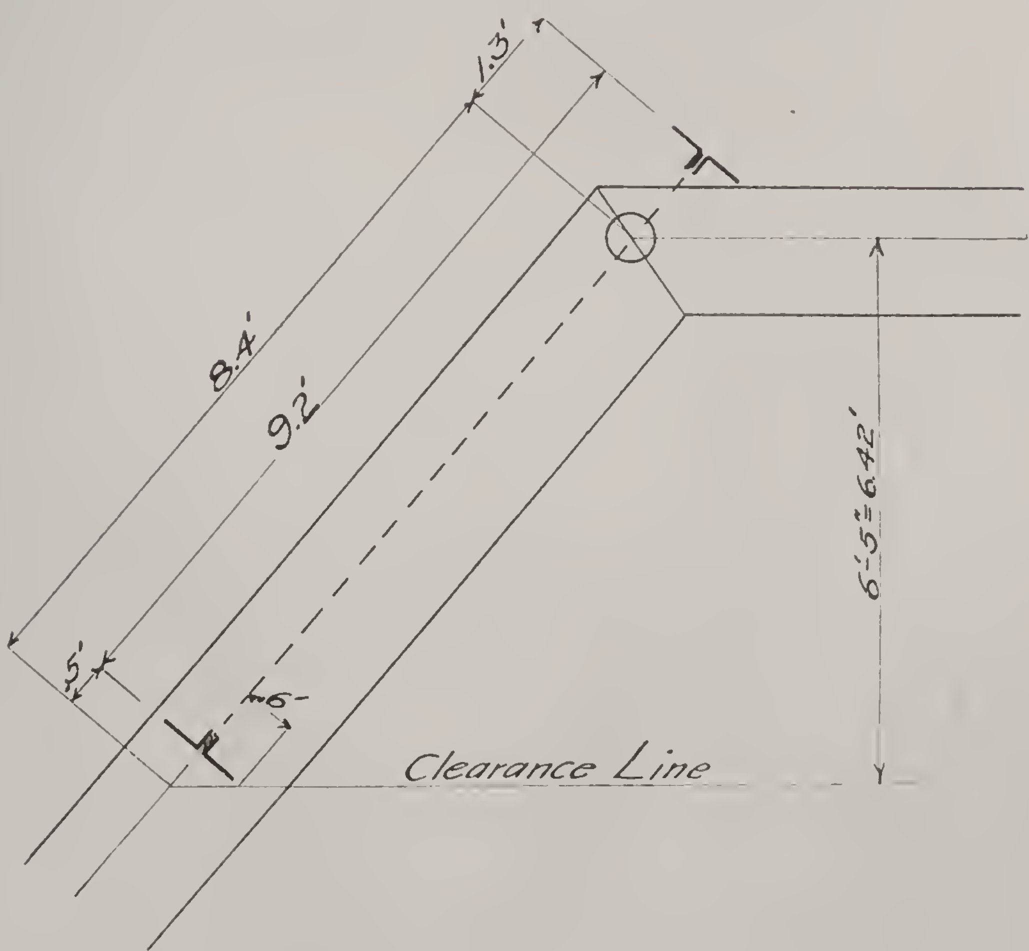


Fig. 61.

From Fig. 55 we see that we can make the vertical distance from the upper clearance line to the upper pin center 6 ft. 5 inches.

Fig. 61 illustrates the method of determining the depth of portal which we may use. This may be laid out to scale and the depth scaled. For the purpose of calculating the stresses the depth does not have to be determined closer than the nearest 0.1 ft.

In the following calculations of the stresses in the end posts and portal bracing due to wind, the methods and notation used in Heller's "*Stresses in Structures*," Arts. 153 to 165 inclusive, will be followed. The specifications §106 directs that the portals be latticed and that they be connected rigidly to the end posts and top chords.

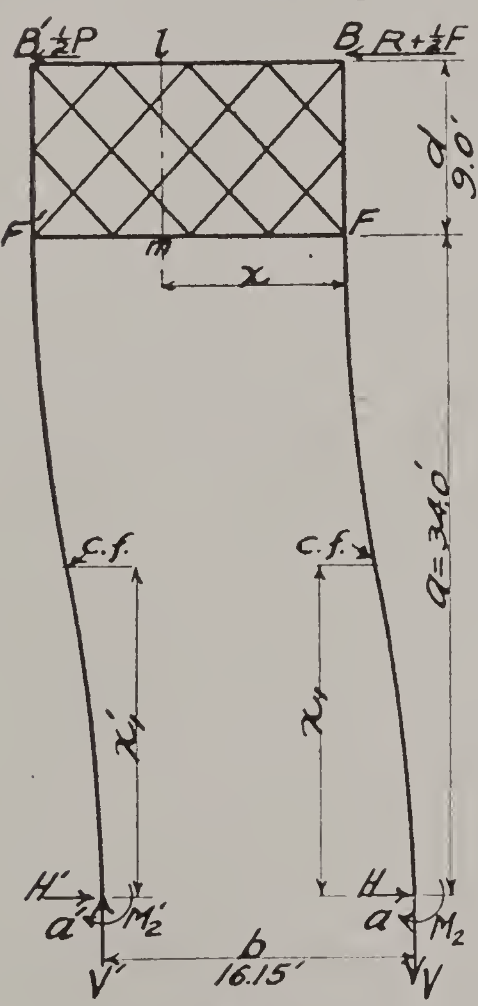


Fig 62.

Fig. 62 gives the general dimensions of the portal bracing and end posts.

First, it must be determined whether the end posts are fixed at the ends by the direct stresses, or not.

From Fig. 60 we may determine approximately that

$k_1 = k_2 =$ about 17 inches.

P=Panel load of wind load on top lateral system=$200 \times 27 = 5400$ pounds.

R=Reaction of top lateral system on the portal=$2P$ (there are 5 panels in the top lateral system.)=10,800 lbs.

To test for the degree of fixity of the ends of the posts assume $H = H' = \frac{1}{2}(R+P)$. (This is assuming fixed ends.) Then $H = H' = 8{,}100$ lbs.

The portal being rigidly connected to the end posts from B to F, fixes the tops of the posts, then the point of contra-flexure would occur midway between a and F or $x_1 = x_1' = 17.0$ ft.

$M_2 = Hx_1 = 8100 \times 17 = 137{,}700$ ft. lbs.=1,652,400 in. lbs. With moments about a point of contra-flexure we get,

$$V = -V' = \frac{1}{b}(R+P)(a+d-x_1) = \frac{1}{16.15} \times 16200 \times (43.0-17.0)$$
$$= 26{,}100 \text{ lbs.}$$

The maximum stress in the end posts occurs in the leeward post when the live load is on the bridge and when the wind is acting.

Dead Load stress	=123,100 lbs.
Live Load stress	=255,400 lbs.
Overturning tendency due to wind on train	= 31,300 lbs.
V due to wind on top lateral system	= 26,100 lbs.
Total maximum direct stress	=435,900 lbs.

The concurrent direct stress in the windward post will be 123,100+255,400—31,300—26,100=321,100 lbs.

The moment of the direct stress at the bottom of the windward post tending to fix the end is

$$\tfrac{1}{2}k_2D = 321{,}100 \times \tfrac{1}{2} \times 17 = 2{,}729{,}400 \text{ in. lbs.}$$

As this is greater than the moment M_2 tending to rotate the post at a, the posts will be fixed at the bottoms.

The maximum bending moment in the post occurs either at a' or at F', and is 1,652,400 in. lbs.

We will try for the end posts the same section as was used for top chord sections CD and DD.

The moment of inertia must be calculated for a vertical axis through the center of gravity.

Area of cross section=51.86 sq. in. Eccentricity=1.68 in.
I (horizontal axis)= 2494 r (horizontal axis)=6.93
I (vertical axis)=3848

The average allowed unit stress for dead and live loads from equation (21)

$$s_w = \frac{17000-90\dfrac{L}{r}}{1+\dfrac{255400}{378500}} = \frac{10460}{1.675} = 6245 \text{ lbs. per sq. in.}$$

When wind stresses are added to the dead and live load stresses this unit may be increased 30%, (Spec. §39), making it 8120 lbs. per sq. in.

For this cross section the actual maximum fiber stress would be (58)

$$s_{\max}=\frac{P}{A}+\frac{Mv}{I}=\frac{435900}{51.86}+\frac{1652400\times 12.875}{3848}$$
$$=8405+5528=13933 \text{ lbs. per sq. in.}$$

Therefore the section must be increased.

As the moment of inertia increases when the area is increased, we may arrive at an approximate figure for the area by considering the equation above as follows:

$$8405 : s_w = 13933 : 8120 \text{ from which } s_w = 4900 \text{ lbs.}$$

This value of s_w assumes that in changing the section we have not changed the radius of gyration, and of course can be used only as a general guide.

Using this value of s_w we find that the approximate required area will be $\frac{435900}{4900}=89$ sq. in. about.

It will be found by trial that this area cannot be made up without materially reducing the radius of gyration, and consequently the allowed unit stress, unless the width or depth of the section be increased.

If the width were incresaed it would necessitate an equal increase in width of the top chord sections and add materially to their weight without increasing their efficiency. (See Spec. §§80 and 100), but the depth of the end posts may be increased somewhat without changing any of the top chord sizes.

After several trials the following section was chosen:

1 Cover plate	24″×⅝″	=15.00 sq. in.
2 Web plates	21″×⅞″	=36.75 sq. in.
2 Top angles	3″×3″×⅝″	= 6.72 sq. in.
2 Bot. angles	4″×3″×⅝″	= 7.98 sq. in.
2 Side plates	15″×⅝″	=18.75 sq. in.
	Total	=85.20 sq. in.

The properties of this section are as follows:

Eccentricity=1.77 in. Area=85.20 sq. in.

I (horizontal axis)=4698 I (vertical axis)=6424

r (horizontal axis)=7.43 in.

Allowed D. L. unit stress$=17000-90\frac{L}{r}=10913$ lbs. per sq. in.

Allowed unit stress for D. L.+L. L. from Eq. (21)= 6516 lbs. per sq. in.

Allowed unit stress for D. L. +L. L.+Wind=8468 lbs. per sq. in.

$$\text{Max. extreme fiber stress}=\frac{435900}{85.20}+\frac{1652400\times12.875}{6424}$$

$$=5116+3312=8428 \text{ lbs. per sq. in.}$$

67. The Portal Bracing. The maximum stresses in the portal bracing will occur when there is no live load on the bridge.

The direct stress then in the leeward post, assuming fixed ends as above, is as follows:

Dead Load stress=123,100 lbs.
V= 26,100 lbs.
Total=149,200 lbs.

The concurrent direct stress in the windward post=123,100 —26,100=97,000 lbs.

The moment of the direct stress at the bottom of the leeward post tending to fix that end is

$$\tfrac{1}{2}k_2D=149{,}200\times\tfrac{1}{2}\times17=1{,}268{,}200 \text{ in. lbs.}$$

The moment M_2 required to fix that end is 1,652,400 in. lbs., therefore the posts are only partially fixed at the bottoms, the tops are fixed by the construction. An approximate mean value of x_1 and x'_1 may be gotten from the equation

$$x_m=\frac{\frac{1}{2}k_2D}{\frac{1}{2}(R+P)}$$

Neglecting V in the value of D we get

$$x_m=\frac{8.5\times123100}{8100}=129 \text{ in.}=10.75 \text{ ft.}$$

We may now get an approximate value for V.

$$\text{1st Approx. } V=-V'=\frac{1}{b}(R+P)(a+d-x_m)$$

$$=\frac{1}{16.15}\times16{,}200(43-10.75)=32{,}300 \text{ lbs.}$$

2nd Approx, D =123,100—32,300= 90,800 lbs.
2nd Approx. D' =123,100+32,300=155,400 lbs.
2nd Approx. M_2 =8.5× 90,800= 771,800 in. lbs.
2nd Approx. M_2' =8.5×155,400=1,320,900 in. lbs.

$$\text{2nd Approx. } H' =\tfrac{1}{2}\left[R+P+\frac{3}{a\,2}(M_2'-M_2)\right]$$

$$=\tfrac{1}{2}\left(16{,}200+\frac{3}{2\times 34\times 12}\times 549{,}100\right)=9110 \text{ lbs.}$$

2nd Approx. $H \quad =R+P-H'=7090$ lbs.

2nd Approx. $x_1 \quad =\dfrac{M_2}{H}=\dfrac{771800}{7090}=109$ in.$=$ 9.08 ft.

2nd Approx. $x'_1 \quad =\dfrac{M_2'}{H'}=\dfrac{1320900}{9110}=145$ in.$=12.08$ ft.

2nd Approx. $V=-V'=\dfrac{1}{b}\left[(R+P)(a+d-x_1)-H'(x'_1-x_1)\right]$

$$=\frac{1}{16.15}\left[16{,}200(43-9.08)-9{,}110\times 3\right]=32{,}330 \text{ lbs.}$$

As this value of V does not differ materially from the first approximation the values of the other quantities are also determined closely enough.

Taking a section lm through the portal and a center of moments at the bottom flange

$$\text{Stress in top flange}=R+\tfrac{1}{2}P+H\frac{a-x_1}{d}-V\frac{x}{d}$$

$$=10800+2700+7090\frac{24.92}{9.0}-32{,}330\frac{x}{9}$$

The maximum compression in the top flange then will occur where $x=0$ and the maximum tension where $x=b$.

Max. Comp. in top flange$=33{,}100$ lbs. $(+)$

Max. Tens. in top flange$=24{,}900$ lbs. $(-)$

Taking the same section and a center of moments in the top flange

$$\text{Stress in bottom flange of portal}=H\frac{a+d-x_1}{d}-V\frac{x}{d}$$

The maximum tension occurs where $x=0$ and the maximum compression where $x=b$.

Max. Tens. in Bot. Flange$=7090\dfrac{33.92}{9}=26{,}700$ lbs. $(-)$

Max. Comp. in Bot. Flange$=26700-32330\dfrac{16.15}{9}=31{,}300$ lbs. $(+)$

The maximum shear at any section is $V=32{,}330$ lbs.

For the portal flanges the least allowable radius of gyration is $\dfrac{1}{120}\times 14\times 12=1.4$ inches. (Spec. §35).

This radius of gyration is taken perpendicular to the plane of the portal.

Try 2 Angles $5''\text{x}3''\text{x}\frac{5}{16}''$, 5 inch legs outstanding.

Area$=2\times2.41=4.82$ sq. in. gross. $r=2.47$ in.

The allowed unit stress$=13{,}000-60\frac{L}{r}=8920$ lbs. per sq. in.

Reqd. area$=\frac{33100}{8920}=3.71$ sq. in.

These are large enough for the maximum compression stress.

The required net area for tension$=\frac{26700}{18000}=1.48$ sq. in.

Actual net area$=4.82-2\times\frac{5}{16}\times1=4.20$ sq. in. ($\frac{7}{8}''$ Rivets.)

For the lattice of the portals we will use angles spaced about as shown in Fig. 62.

Any vertical section will cut four lattice angles, so we will consider the shear as equally divided among them.

The secant of the angle of inclination of the lattice is about 1.4, so the stress in each lattice angle$=1.4\times\frac{32330}{4}=11{,}300$ lbs. tension or compression.

The smallest angles allowable to use are $3''\text{x}2\frac{1}{2}''\text{x}\frac{5}{16}''$, (See Spec. §83), which will be ample to take the above stress.

68. Design of Floor Beams. The weight of the beam itself is a uniform load, the weights of the floor, stringers and live load form two concentrated loads on the floor beam 6′-6″ apart. Since the beam's own weight is a very small proportion of the total load, it will be considered as concentrated at the stringer connections also.

The maximum live load concentration on the beam may be taken from the specifications, Table I, or calculated from the wheel loads.

Live load at each stringer connection	$=80{,}000$ lbs.
Dead load from floor $=200\times27=$	5,400 lbs.
Weight of stringers $=165\times27=$	4,455 lbs.
Weight of $5\frac{1}{2}$ ft. of floor beam say $=$	945 lbs.
Total Dead Load	$=10{,}800$ lbs.

at each stringer.

The distance center to center of trusses is 16 ft. 1¾ in. This is considered as the distance between supports of the floor beams. The distance between the center of the truss and the nearest stringer connection is 4′-9⅞″.

The moments on the floor beam are:

Dead load moment=10,800×4.82= 52,060 ft. lbs.

Live load moment=80,000×4.82=385,600 ft. lbs.

Economic depth from equation (10) (no flange plates) is

$$1.52\sqrt{\frac{411600\times12}{10000\times\frac{3}{8}}}=55 \text{ inches}$$

The depth assumed in the calculations for the depth of truss was about 13 inches more than the depth of the stringer, or 64¼ inches. (See Fig. 55).

It is desirable to have the stringer connection come between the flange angles of the floor beam rather than to have it lap over the vertical legs of these angles, in order to dispense with filler plates under the connectian angles. For these reasons then we will use a web plate 64 inches deep.

The maximum shear=10,800+80,000=90,800 lbs.

Using a ⅜″ web plate the unit shear$=\frac{90800}{24}=$3780 lbs. per sq. in., which is safe.

Assuming that the flange angles will be 6″x6″, the effective depth will be about 61 inches.

$$\text{Approx. D. L. flange stress}=\frac{52060\times12}{61}=10{,}240 \text{ lbs.}$$

$$\text{Approx. L. L. flange stress}=\frac{385600\times12}{61}=75{,}860 \text{ lbs.}$$

$$\text{Approx. D. L. required area}=\frac{10240}{20000}=0.52 \text{ sq. in.}$$

$$\text{Approx. L. L. required area}=\frac{75860}{10000}=7.59 \text{ sq. in.}$$

Total=8.11 sq. in.

2*Ls* 6″x6″x$\frac{7}{16}$″=10.12−4×$\frac{7}{16}$×1=8.37 sq. in. net. (⅞ inch rivets, no holes out of horizontol legs, two holes out of vertical legs, to comply with Spec. §64.)

The actual effective depth=64.25−2×1.66=60.93 in.

This will not change the flange stresses given above appreciably, so the flange as designed will answer.

There must be sufficient rivets through the flange angles and web to develop the entire flange stress between the end of the beam and the stringer connection. This will require more rivets than would be given by the horizontal increment (49) because the flange angles cannot run to the theoretical end of the beam which is at the center of the truss.

End Floor Beams are required by §10 of the specifications, and it is always good practice to use them rather than to allow the end stringers to rest directly on the abutments.

The end floor beam must carry the half panel load from the end panel of the bridge, and also the load from the short space between the end floor beam and the back wall, which is usually bridged by a cantilever bracket riveted to the beam opposite each stringer. This space will be about two feet in our case.

The dead load at each stringer connection will be

Floor	$(13.5+2.0)200=3100$ lbs.
Stringer	$15.5\times165=2560$ lbs.
Floor Beam say	$=\ 840$ lbs.
	Total D. L. $=6500$ lbs.

The live load reaction at the stringer connection must be determined from the actual wheel loads. The maximum reaction on the end floor beam will occur with the wheels placed as shown in Fig. 63.

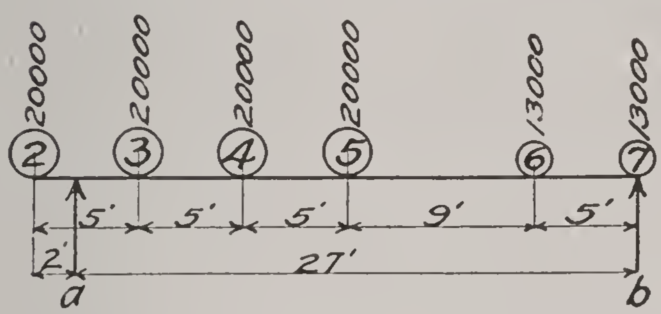


Fig. 63.

The reaction at $a=\dfrac{1785000}{27}=66,100$ lbs.

Dead load moment $=\ 6,500\times4.82=\ 31,300$ ft. lbs.

Live load moment $=66,100\times4.82=318,600$ ft. lbs.

In order to simplify the connection of the end floor beam to the truss, it should be made as shallow as is consistent. We will therefore make the depth only sufficient to allow the stringer to enter between the horizontal legs of the flange angles. This will require a depth of about 52½ or 53 inches.

We will use a web plate 53″x⅜″.

Unit shearing stress $=\dfrac{72600}{19.87}=3660$ lbs. per sq. in., which is safe.

The effective depth will be about 51 inches.

Approx. D. L. flange stress$=\frac{31300\times 12}{51}=$ 7,360 lbs.

Approx. L. L. flange stress$=\frac{318600\times 12}{51}=$75,000 lbs.

Approx. required D. L. area$=\frac{7360}{20000}=$0.37 sq. in.

Approx. required L. L. area$=\frac{75000}{10000}=$7.50 sq. in.

Total=7.87 sq. in.

If the required rivet pitch (49) is not too small for a single line of rivets in the flange, we may use unequal legged angles for the flange with the long legs horizontal, which will be more economical.

Try 2*Ls* 6″x3½″x½″. Net area$=9.00-2\times\frac{1}{2}\times 1=8.00$ sq. in. (⅞ inch rivets and no holes out of horizontal legs.)

Actual effective depth$=53.25-2\times 0.83=51.59$.

Actual D. L. flange stress$=\frac{31300\times 12}{51.59}=$ 7,300 lbs.

Actual L. L. flange stress$=\frac{318600\times 12}{51.59}=$74,100 lbs.

Actual required D. L. area$=\frac{7300}{20000}=$0.37 sq. in.

Actual required L. L. area$=\frac{74100}{10000}=$7.41 sq. in.

Total=7.78 sq. in.

This will not allow a reduction below the section assumed above.

The number of rivets required to connect the flange angles to the web, between the stringer connection and the end of the beam will be the total flange stress 81,400 lbs. divided by the value of a ⅞″ rivet in bearing on the ⅜″ web. (See Spec. §40).

Number of rivets$=\frac{81400}{3938}=$21.

The distance from the stringer connection to the end of the beam will be about 3′-9″.

Required rivet pitch$=\frac{45}{12}=$2.12 inches, which is less than should be allowed in a single line (7). (Spec. §54). Therefore

6″x6″ angles must be used for the flange angles.

Try 2*Ls* 6″x6″x$\frac{7}{16}$″. Net area$=10.12-4\times\frac{7}{16}\times1=8.37$ sq. in. Actual effective depth$=53.25-2\times1.66=49.93$ in.

Actual D. L. flange stress$=\dfrac{31300\times12}{49.93}=$ 7,500 lbs.

Actual L. L. flange stress$=\dfrac{318600\times12}{49.93}=$76,600 lbs.

Actual required D. L. area$=\dfrac{7500}{20000}=0.38$ sq. in.

Actual required L. L. area$=\dfrac{76600}{10000}=7.66$ sq. in.

Total$=8.04$ sq. in.

Use for flanges 2*Ls* 6″x6″x$\frac{7}{16}$″.

69. Top Lateral Bracing. (59) The top lateral system is a horizontal Pratt truss of five panels, with the portals at the ends acting as abutments.

Panel load of wind load for top lateral system$=200\times27=$ 5400 lbs. $Sec\theta=\dfrac{31.45}{16.15}=1.95$.

The stresses are as follows:

Diagonal $BC=2\times5400\times1.95=21{,}100$ lbs.
Diagonal $CD=1\times5400\times1.95=10{,}600$ lbs.
Diagonal $DD=0$
Strut $CC=1\frac{1}{2}\times5400=8{,}100$ lbs.
Strut $DD=\frac{1}{2}\times5400=2{,}700$ lbs.

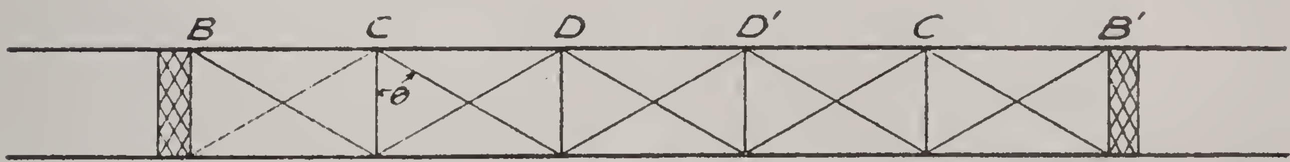


Fig. 64.

Required area for diagonal $BC=\dfrac{21100}{18000}=1.18$ sq. in.

To comply with specifications §11 the lateral bracing must be made of shapes capable of resisting compression. It is not good practice to use angles smaller than about 3½″x3″x$\frac{5}{16}$″ for these laterals. The net area of one angle 3½″x3″x$\frac{5}{16}$″$=1.94-2\times\frac{5}{16}\times1=1.32$ sq. in., so that these angles will answer for all the diagonals.

The size of the intermediate struts will be determined by §§35, 83 and 107 of the specifications. The unsupported length will be about 170 in. From Spec. §35 the least allowable radius of gyration$=\frac{170}{120}=1.42$ inches. This will require at least $2Ls$ $3''\text{x}2\frac{1}{2}''\text{x}\frac{5}{16}''$, which are also required to comply with §83 of the specifications.

$$\text{Allowed unit stress}=13{,}000-60\frac{L}{r}=6{,}000 \text{ lbs. per sq. in.}$$

$$\text{Required area}=\frac{8100}{6000}=1.35 \text{ sq. in.}$$

$$\text{Actual area}=2\times 1.63=3.26 \text{ sq. in.}$$

These top strut angles are run over the top chords and riveted to the cover plates, and two other angles back to back are riveted between the intermediate posts as low down as the specified head room will allow (See Spec. §107). These two struts are connected by diagonal lattice work of angles similar to the portal (See Fig. 62).

70. Bottom Lateral Bracing. The bottom lateral system (Fig. 64) must resist a static load of 150 lbs. per lin. ft. and a moving load of 450 lbs. per lin. ft. (Spec. §24.)

Panel load D. L. wind$=150\times 27=$ 4,050 lbs.

Panel load L. L. wind$=450\times 27=12{,}150$ lbs.

$sec\theta=1.95$ (same as for top lateral system).

The total stresses in the diagonals are as follows:

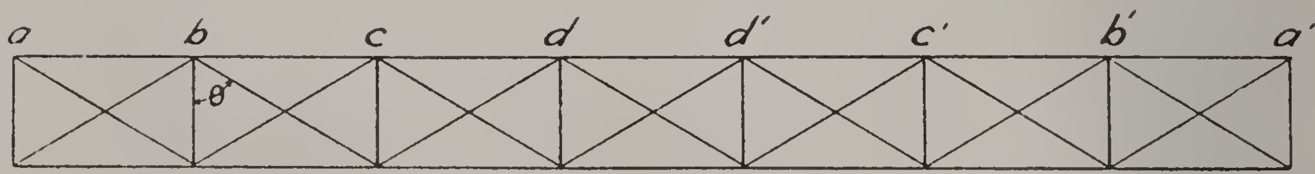


Fig. 65.

Diag. $ab=4050\times 3\times 1.95+12{,}150\times 3\times 1.95=94{,}700$ lbs.

Diag. $bc=4050\times 2\times 1.95+12{,}150\times \frac{15}{7}\times 1.95=66{,}500$ lbs.

Diag. $cd=4050\times 1\times 1.95+12{,}150\times \frac{10}{7}\times 1.95=41{,}700$ lbs.

Diag. $dd=4050\times 0\times 1.95+12{,}150\times \frac{6}{7}\times 1.95=20{,}300$ lbs.

$$\text{Required area } ab=\frac{94700}{18000}=5.26 \text{ sq. in.}$$

$$\text{Required area } bc=\frac{66500}{18000}=3.70 \text{ sq. in.}$$

Required area $cd=\frac{41700}{18000}=2.32$ sq. in.

Required area $dd'=\frac{20300}{18000}=1.13$ sq. in.

To comply with §33 of the specifications both legs of an angle in tension must be connected if the area of both legs is regarded as effective section, and therefore according to specifications §64 at least two holes must be deducted from the gross section of each angle.

Use for diagonal ab $2Ls$ $6''\text{x}3\frac{1}{2}''\text{x}\frac{3}{8}''$. Net area$=6.86-4\times\frac{3}{8}\times1=5.36$ sq. in.

Use for diagonal bc $2Ls$ $5''\text{x}3\frac{1}{2}''\text{x}\frac{5}{16}''$. Net area$=5.12-4\times\frac{5}{16}\times1=3.87$ sq. in.

Use for diagonal cd $1L$ $5''\text{x}4''\text{x}\frac{3}{8}''$. Net area$=3.24-2\times\frac{3}{8}\times1=2.49$ sq. in.

Use for diagonal dd' $1L$ $3\frac{1}{2}''\text{x}3''\text{x}\frac{5}{16}''$. Net area$=1.94-2\times\frac{5}{16}\times1=1.32$ sq. in.

The bottom flanges of the floor beams act as the bottom lateral struts, and the compression from the lateral forces tends to relive the tension in them from vertical loads.

71. Shoes and Rollers. The end reaction will be $3\frac{1}{2}$ panel loads of $D.L.+L.L.=3\frac{1}{2}\times96{,}430=337{,}500$ lbs.

According to specifications §113 this will require

$$\frac{337500}{250}=1350 \text{ sq. in. bearing on the masonry.}$$

The masonry plate may be made say $3'\text{-}6''$ long by 33 inches wide, giving a bearing area of 1386 sq. in. According to specifications §114 the rollers cannot be made less than $5\frac{3}{4}$ inches in diameter.

The maximum allowed pressure on the rollers will be $300\times5\frac{3}{4}=1725$ lbs. per lin. in.

$$\text{Required length of rollers}=\frac{337500}{1725}=196 \text{ inches.}$$

This might be made up of 6 rollers 33 inches long, or 7 rollers 28 inches long. The details can not be worked out without detailing the end posts, end floor beams and shoes.

72. Estimate and Stress Sheet. The *estimate* of weight will now be given. The details can only be estimated approximately until the detail drawings are made. Ordinarily the

details are put in the estimate as a percentage of the main truss members, and an estimator of experience in detailing and estimating can choose his percentages so that the total error in weight will be very small.

The *stress sheet* may now be drawn up. (See Fig. 66.)

This is usually as much as is done until after the contract is awarded. The bridge company who fabricates the work makes the detail shop drawings, which are then approved by the railroad company's engineer.

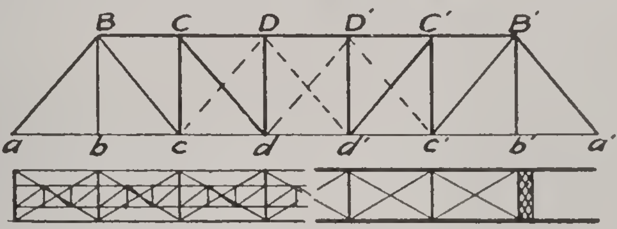
FORM No. 1

THE OHIO STATE BRIDGE COMPANY

Sheet No. 1 | Made by C.T.M. | Date 12-10-08

Estimate for A.&T.P.Ry. Bridge No. 107 Southern Div.

Span Extreme 192.5 ft.	Span C. to C. 189.0'	Estimated DL per ft. { Steel 1923	Total Steel 366.250
Roadway Single Track	7 Panels at 27.0'	Floor & Track 400	Steel per ft. 1936
Sidewalk None	Depth C. to C. 32.0'	Total 2323	Total Lumber
Capacity Trusses E-40	Length of Diag. 41.87	Panel Load per Truss DL. 31360	
Capacity Floor See Spec.	Width c. to c. 16.15'	" " " LL. 65070	
Specifications Cooper's '06	Sec. $\frac{41.87}{32}$ Tg. $\frac{27}{32}$		



Stringers 6'-6" c. to c.
Ties 8"×8"×10'-0"
Guards 6"×8"

MEM	DL Stress	LL Stress	~~Impact~~ Wind	Total Stress	Unit Stress	Req. Area	MATERIAL	Actual Area	No. Pcs.	Wt. P. Ft.	Length	WEIGHT
							1-24"×5/8"		4	51.0	42.5	8670
	Wind bending = 137700'#						2-21"×7/8"		8	44.6	42.0	15000
aB	+123.1	+255.4	±57.4				2-15"×5/8"	85.2	8	31.9	42.0	10700
							2∟3"×3"×5/8"		8	11.5	42.5	3910
							2∟4"×3"×5/8"		8	13.6	42.0	4570
Bc	-82.1	-182.4				22.35	4-6"×15/16" (Eq)	22.5	16	19.1	46.5	14200
Cd	-41.0	-121.6				14.21	2-6"×1 3/16" "	14.25	8	24.2	46.5	9000
Dd'	0	-73.0				7.3	2-4"×15/16" Adj. "	7.50	8	12.8	49.5	5080
D'c'	+41.0	-36.5				2.01	1 Sq. 1 7/16" Adj. "	2.07	4	7.0	49.5	1390
Bb	-20.9	-80.1				11.32	2[12"×25# [ ] Net	11.39	8	25.0	33.0	6600
Cc	+41.8	+93.0				21.41	2[15"×40#	23.52	8	40.0	34.0	10880
Dd	+10.5	+55.8				13.27	2[12"×25#	14.70	8	25.0	34.0	6800
abc	-79.4	-164.7	-101.8			22.29	4∟6"×3½"×½" Net	23.0	32	15.3	26.0	12730
							2-14"×3/8"		16	17.9	26.0	7450
cd	-132.3	-274.5	-169.7			37.14	2-7"×1 5/16"	37.64	8	31.2	30.5	7600
							2-7"×1 3/8"		8	32.7	30.5	7980
dd'	-158.8	-329.5	-203.6			44.58	2-7"×1 5/8"	44.64	4	38.7	30.5	4720
							2-7"×1 9/16"		4	37.2	30.5	4540
							1-24"×½"		4	40.8	26.0	4240
BC	+132.3	+274.5				43.13	2-18"×9/16"	40.73	8	34.4	26.0	7160
							2∟3"×3"×3/8"		8	7.2	26.0	1500
							2∟4"×3"×9/16"		8	12.4	26.0	2580
									Forward →			157300

FORM No. 2

THE OHIO STATE BRIDGE COMPANY

Sheet No. 2 Made by Date

Estimate for A.&T.P. Ry. Bridge No. 107 Southern Dir.

MEM	DL Stress	LL Stress	Impact	Total Stress	Unit Stress	Req. Area	MATERIAL	Actual Area	No. Pcs.	Wt. P. Ft.	Length	WEIGHT
								Brot. Fowd.				157300
							1-24"×1/2"		2	40.8	83.0	6780
CD &	+158.8	+329.5				51.76	2≐18"×3/4"	51.86	4	45.9	83.0	15240
DD							2∟3"×3"×7/16"		4	8.3	83.0	2760
							2∟4"×3"×5/8"		4	13.5	83.0	4520
												186600
							Truss	Details	25%			46650
							Total Truss Members →					233250
								233250				
4 Shoes		Bearing Area = 1350 □"							4	@2000#		8000
	End Reaction = 337500#											
2 Roller Nests							5¾" Rollers 196 lin. in.		2	@1500#		3000
								11000				
6 Intermediate Floor Beams												
	DL. Mom. = 520600'#				Web		1-64"×3/8"		6	81.6	15.3	7500
	LL. " = 385600'#				Flgs.	8.11	2∟6"×6"×7/16" # Net	8.37	24	17.2	15.3	6320
	Shear = 90800#				Web Spl.		2≐12"×3/8"		24	15.3	4.3	1580
				Conn. ∟s			2∟6"×3½"×1/2"		24	15.3	6.0	2200
				Fills			2≐3½"×7/16"		24	5.2	5.0	620
				Knees			1-12"×3/8" (Av.)		12	15.3	2.0	370
				"			2∟6"×3½"×3/8"		24	11.7	2.0	560
									Rivets	4%		760
								19910				
2 End Floor Beams												
	DL. Mom. = 313000'#				Web		1-53"×3/8"		2	67.6	15.3	2070
	LL. " = 318600'#				Flgs.	8.42	2∟6"×6"×1/2" # Net	9.50	8	19.6	15.3	2400
	End Shear = 72600#						Details	&	Riv.	50%		2230
								6700				
14 Stringers					Web		1-51"×3/8"		14	65.0	27.0	24600
	DL. Mom. = 328000'#				Flgs.	8.75	2∟6"×3½"×9/16" # Net	8.93	56	17.1	27.0	25850
	LL. " = 344600'#				Conn.		2∟6"×6"×1/2"		56	19.6	4.2	4610
	End Shear = 64200#				Fills		2≐12"×9/16"		56	23.0	3.7	4760
					Stiff.		2∟3"×3"×3/8" (Crimp)		168	7.2	4.2	5090
									Rivets	4%		2590
								67500				
Stringer Laterals					Diag.		1∟3½"×3"×3/8"		21	7.8	10.0	1640
					Struts		1 " " "		14	"	6.0	660
					Conn.		1-12"×3/8" (Av.)		42	15.3	2.5	1600
									Rivets	3%		120
								4020	Forward →			342380

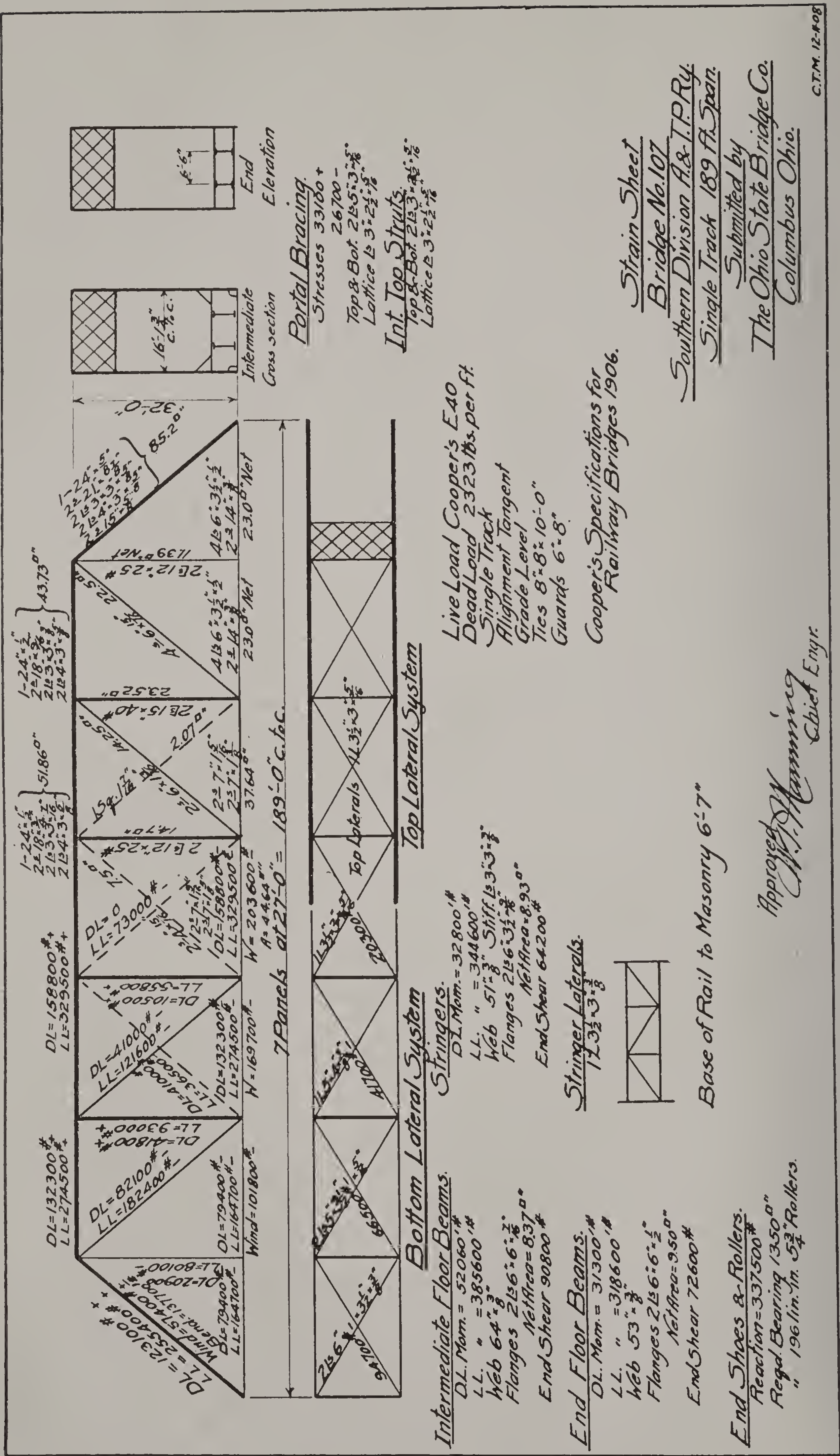
FORM No. 2

THE OHIO STATE BRIDGE COMPANY

Sheet No. 3 Made by Date

Estimate for A.&T.P. Ry. Bridge No. 107 Southern Div.

MEM	DL Stress	LL Stress	Impact	Total Stress	Unit Stress	Req. Area	MATERIAL	Actual Area	No. Pcs.	Wt. P. Ft.	Length	WEIGHT
								Brot. Fowd. →				342380
Bottom Laterals												
ab				94.7	18.0	5.26	2 Ls 6" × 3½" × 3/8" Jt. Net	5.36	8	11.7	30.0	2800
bc				66.5	"	3.70	2 Ls 5" × 3½" × 5/16" " "	3.87	8	8.7	30.0	2090
cd				41.7	"	2.32	1 L 5" × 4" × 3/8" J "	2.49	4	11.0	30.0	1320
dd				20.3	"	1.13	1 L 3½" × 3" × 5/16" " "	1.32	2	6.6	30.0	400
				Conn. Pls			1-36" × 3/8" (Av.)		12	45.9	4.0	2200
				"	"		" " "		4	"	3.0	550
				Spl.	"		1-14" × 1/2"		2	23.8	6.0	290
				"	"		1-12" × 3/8"		5	15.3	6.0	460
				Conn. Ls			1 L 3½" × 3" × 5/16"		18	6.6	1.5	180
									Rivets		3%	310
								10600				
Top Laterals							1 L 3½" × 3" × 5/16"		10	6.6	32.0	2110
								Details & Riv.			10%	210
								2320				
2 Portal Struts				+33100#			4 Ls 5" × 3" × 5/16"					
				−26700#			Lattice 1 L 3" × 2½" × 5/16"		2 @ 2000#			4000
4 Intermediate Struts							4 Ls 3" × 2½" × 5/16"					
							Lattice 1 L 3" × 2½" × 5/16"		4 @ 1500#			6000
							Track bolts & Anchors		−	5	189	950
								Total Steel →				366250



Intermediate Floor Beams.
End Floor Beams.
End Shoes & Rollers.
Bottom Lateral System
Stringers.
Stringer Laterals.
7 Panels at 27'-0" = 189'-0" c. to c.
Top Laterals
Top Lateral System
Base of Rail to Masonry 6'-7"
Live Load Cooper's E40
Single Track
Alignment Tangent
Grade Level
Ties 8"×8"×10'-0"
Guards 6"×8"
Cooper's Specifications for Railway Bridges 1906.
Approved
Chief Engr.
Intermediate Cross section
End Elevation
Portal Bracing.
Int. Top Struts.
Strain Sheet
Bridge No. 107
Southern Division A.&T.P.Ry.
Single Track 189 ft. Span.
Submitted by
The Ohio State Bridge Co.
Columbus Ohio.
C.T.M. 12-4-08

Fig. 66.

CHAPTER VII.

DETAILS OF PIN CONNECTED BRIDGES.

The design of the bridge and stress sheet are usually worked out by the purchaser's engineer and submitted to the prospective bidders for prices, but sometimes the bidders are asked to submit designs with their bids. (16)

After the contract is awarded the detail shop drawings are made by the contractor and approved by the purchaser's engineer. (See Chapter III.) These detail drawings show the sizes and positions of all connections and details of members, together with the number and location of all rivets.

The details must be so proportioned that the stresses will be safely and economically transmitted from member to member and finally to the abutments.

73. Pins. A pin is a beam which transmits the stresses at a joint. It is acted upon by forces in different planes, which produce bending moments and shears in it.

It is usually convenient to resolve these forces into their vertical and horizontal components and get the bending moments in these two planes separately. *The maximum bending moment* at any point then is the resultant of the horizontal and vertical moments at that point. Likewise *the maximum shear* at any point is the resultant of the horizontal and vertical shears at the point.

Since, in most cases, the maximum stresses in all of the members connecting to a pin do not occur under the same loading, the condition for a maximum moment in the pin is uncertain, and the moment must be calculated for the several conditions which give maximum stresses in the various members.

In proportioning the pin for *shear* it must be remembered that the maximum intensity of shear on any cross section of a solid cylinder is equal to four thirds the average intensity.[1]

The *bearing areas* of the members on the pin must be sufficient so that the material will not crush. (10) On this account it is well to have *large pins,* because the larger the pin the less thickness of pin plates required, and also there will be less danger

[1] See Heller's "Stresses in Structures," Art. 71. Also Rankine's Applied Mechanics, Art. 309.

of unequal distribution of stress to the different parts of a member. For example, if there are four bars in a panel of the lower chord, they should be stressed in proportion to their areas, but this will not occur if the pin should bend so as to relieve some of the bars of stress.

On the other hand, the larger the pins the larger will be the diameters of the eyebar heads, and it is often difficult to find room for them, especially at the hip joint.

The arrangement of the parts of the members on the pin is called *the packing,* and this should be such as to produce as small a moment as possible on the pin while at the same time insuring that the eye bars do not pull out of line in passing from joint to joint, more than about one-eighth of an inch per foot, and that the riveted members are of constant width throughout their length.

The sizes of pins must be found *by trial,* since the moments depend upon the thicknesses of the bearings, and to get these we must first assume a diameter for the pin.

74. Calculation of Pins. A few of the joints of the truss designed in Chapter VI will now be detailed to illustrate the methods.

The Hip Joint. (B). According to §90 of the specifications the least size of pin which may be used here is $0.8\times6=4.8$ inches, or say $4\frac{7}{8}$ inches.

The allowed bearing pressure on one linear inch of this pin is $4\frac{7}{8}\times12{,}500=60{,}940$ lbs. for live loads and 121,880 lbs. for dead loads. (See Spec. §41.)

Required bearing on end post aB

$$DL=\frac{123100}{121880}=1.01 \text{ in.}$$

$$LL=\frac{255400}{60940}=4.19 \text{ in.}$$

$$\text{Total}=5.20 \text{ in.}$$

Required bearing on top chord BC

$$DL=\frac{132300}{121880}=1.09 \text{ in.}$$

$$LL=\frac{274500}{60940}=4.50 \text{ in.}$$

$$\text{Total}=5.59 \text{ in.}$$

Required bearing on hip vertical *Bb*

$$DL = \frac{20900}{121880} = 0.17 \text{ in.}$$

$$LL = \frac{80100}{60940} = 1.32 \text{ in.}$$

$$\text{Total} = 1.49 \text{ in.}$$

The bearing pressures on the eyebars of member *Bc* is taken care of by complying with §90 of the specifications.

With these bearing thicknesses the spacing of the forces acting on the pin may be determined approximately, as shown in Fig. 67. Sufficient clearance must be allowed between the different parts to allow them to be easily assembled.

Fig. 67 shows a horizontal projection of the joint. In order to render the bending moment as small as possible (Spec. §90) the eye bars should be packed as near their resistences as possible.

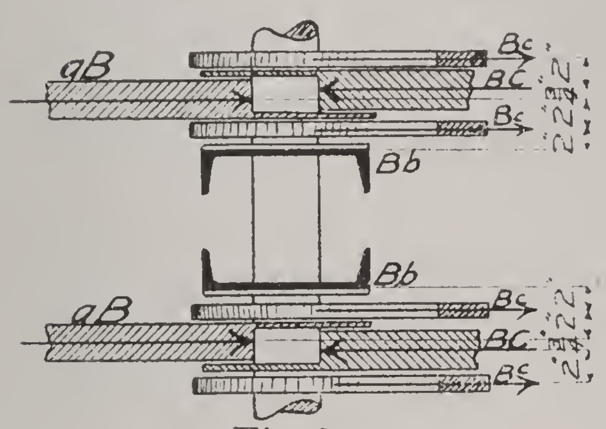


Fig. 67.
PACKING AT B.

The bending moments and shears will have to be calculated for two positions of the live load, one which gives maximum stresses in the top chord end post and hip vertical, and one which gives a maximum stress in the diagonal *Bc*.

The specifications §41 permits the calculation of the moments on the assumption that the pressures are uniformly distributed over the middle half of the bearing areas. The moments will be only slightly increased if we consider the forces as *concentrated at the centers* of the bearing areas, and this will greatly simplify the calculations. This assumption is usually made.

The calculation of the moments is best made in tabular form, remembering that the moment at any force is equal to the moment at the next preceding force plus the product of the shear by the distance between the forces.

In making up the table always begin at the outside where the shear is zero and work toward the center where the shear is again zero.

MAXIMUM STRESSES IN aB AND BC

MOMENTS OF HORIZONTAL COMPONENTS.

Mem.	Horizontal Component	Shear	Lever Arm in inches	Moments Increment	Moments Total
Bc	— 40700				
		— 40700	2	— 81400	
BC	+203400				— 81400
		+162700	$\frac{3}{4}$	+122000	
aB	—122000				+ 40600
		+ 40700	2	+ 81400	
Bc	— 40700				+122000
		000	2	000	
Bb	000				+122000

MOMENTS OF VERTICAL COMPONENTS.

Mem.	Vertical Component	Shear	Lever Arm in inches	Moments Increment	Moments Total
Bc	— 48200				
		—48200	2	— 96400	
BC	000				— 96400
		—48200	$\frac{3}{4}$	— 36150	
aB	+144600				—132550
		+96400	2	+192800	
Bc	— 48200				+ 60250
		+48200	2	+ 96400	
Bb	— 48200				+156650

The maximum for this loading occurs at *Bb* and is $\sqrt{(122000)^2+(156650)^2}$=198,730 in. lbs.

MAXIMUM STRESS IN Bc.

MOMENTS OF HORIZONTAL COMPONENTS.

Mem.	Horizontal Component	Shear	Lever Arm in inches	Moments Increment	Moments Total
Bc	— 42800				
		— 42800	2	— 85600	
BC	+184100				— 85600
		+ 41320	$\frac{3}{4}$	+106000	
aB	— 98500				+ 20400
		+ 42800	2	+ 85600	
Bc	— 42800				+106000
		000	2	000	
Bb	000				+106000

MOMENTS OF VERTICAL COMPONENTS.

Mem.	Vertical Components	Shear	Lever Arm in inches	Moments Increment	Moments Total
Bc	— 50500				
		—50500	2	—101000	—101000
BC	— 000				
		—50500	$\frac{3}{4}$	— 37800	—138800
aB	+116500				
		+66000	2	+132000	— 6800
Bc	— 50500				
		+15500	2	+ 31000	+ 24200
Bb	— 15500				

The maximum resultant moment for this loading occurs at *aB* and is $\sqrt{(20400)^2 + (138800)^2} = 140{,}300$ in. lbs.

Then the maximum bending moment on the pin occurs under full loading and is 198,700 in. lbs.

With an allowed fiber stress of 18,000 lbs. per sq. in. this will require a 4⅞ inch pin, (see Cambria, page 312), or exactly the size that was assumed.

The maximum shear occurs for full load between members *BC* and *aB*, and is $\sqrt{(162700)^2+(48200)^2} = 170{,}000$ lbs.

This gives a maximum unit shear of $\frac{4\times170000}{3\times18.66} = 12150$ lbs. per sq. in.

The specifications only allows a unit shear of 9000 lbs. per sq. in. (see §41) so the size must be increased. This will not change the shears in any way, but will change the required bearing areas and lever arms and moments.

Required area for shear $= \frac{4\times170000}{3\times9000} = 25.2$ sq .in.

A 5¾ inch pin will answer.

In a similar manner the pins at the other joints are figured with the following results:

At *a* a 5¾ inch pin is required.
At *c* a 5½ inch pin is required.
At *d* a 5⅜ inch pin is required.

It will make the shop work somewhat less if these pins are all made the same size, so we will use 5¾ inch pins at *a, c, d* and *B*.

No pin is used at b, but the bottom chord is run through continuous from a to c, and a riveted connection is made at b between Bb and the chord abc.

At C a $4\frac{7}{8}$ inch pin is required, and we will use the same size at D.

Figures 68 to 71, inclusive, show the packing at the various joints.

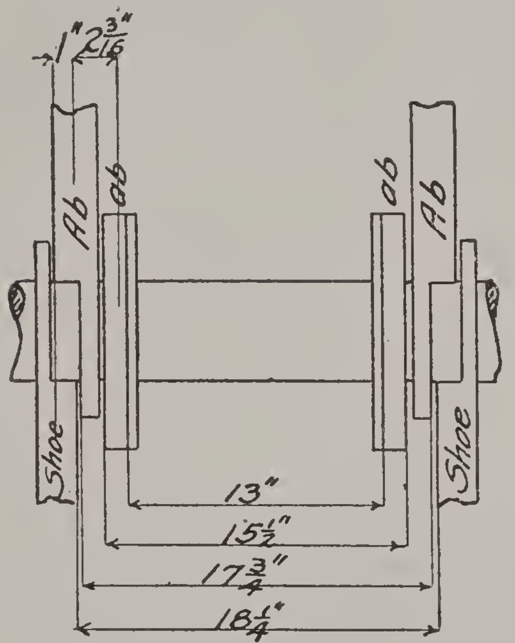


Fig. 68.
Packing at a.

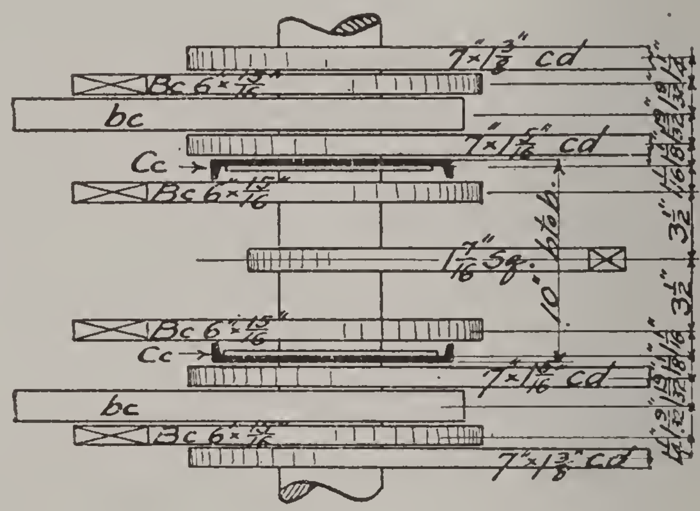


Fig. 69.
Packing at c.

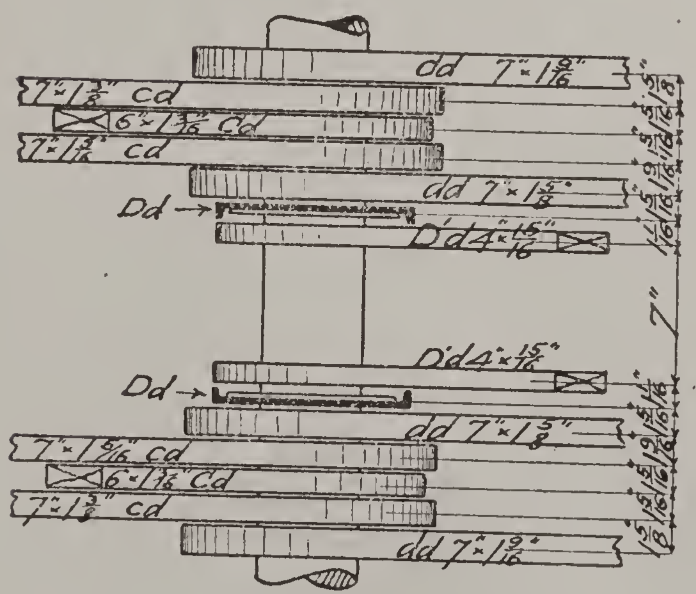


Fig. 70.
Packing at d.

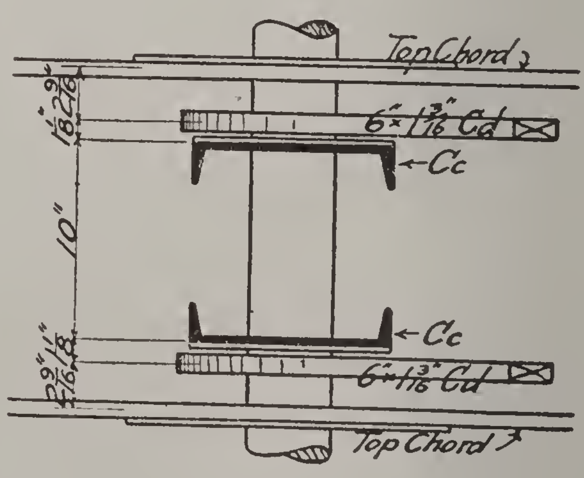


Fig. 71.
Packing at C.

75. Details of a Riveted Tension Member. (11) A detail drawing of one end of the lower chord abc is shown in Fig. 72. The width of the member is determined by the packing at a and c as shown in Figs. 68 and 69.

The maximum stress in the member is 345,900 lbs. ($DL+LL$ +Wind). The average allowed unit stress in tension is 15,520 lbs. per sq. in., and the required net area of the body of the member is 22.29 sq. in. (See Art. 64.) According to the specifications §68 the net section through the pin hole must be one-third in excess of this amount, or 29.72 sq. in., and the least section back of the pin hole 60% of this, or 17.83 sq. in.

The net section of the 2—14″×3/8″ plates through the pin hole is

$$2(2\times 2\tfrac{1}{8}+2\sqrt{(2\tfrac{1}{2})^2+(4\tfrac{7}{8})^2}-5\tfrac{3}{4}-2)\tfrac{3}{8}=5.60 \text{ sq. in.}$$

The balance=29.72—5.60=24.12 sq. in., must be made up of pin plates.

The effective net width through the pin hole of plates 16 inches wide is (See Fig. 72)

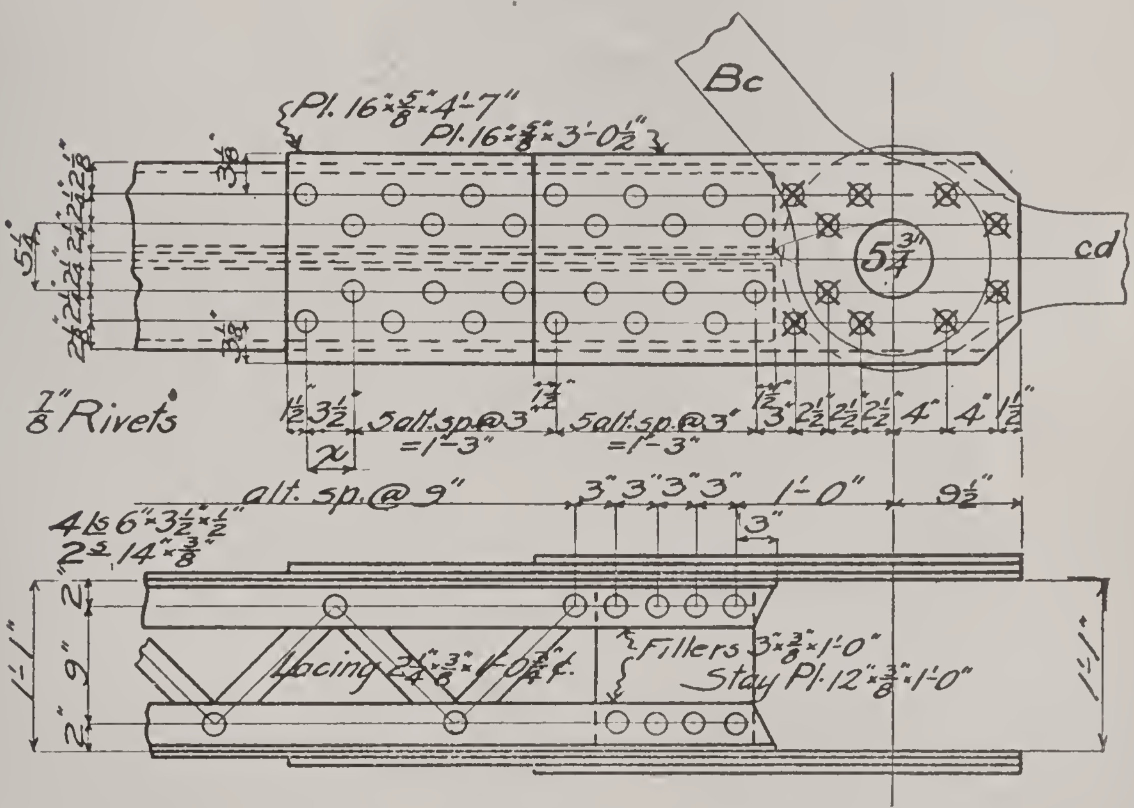


Fig. 72.

$$(2\times 3\tfrac{1}{8}+2\sqrt{(2\tfrac{1}{2})^2+(4\tfrac{7}{8})^2}-5\tfrac{3}{4}-2)=9.46 \text{ in.}$$

Then the required thickness for pin plates 16 inches wide to take the tension is

$$\frac{24.12}{9.46}=2.55 \text{ in., or say } 2\tfrac{1}{2} \text{ inches.}$$

This will require two pin plates on each side 16″x5/8″.

The average allowed unit stress in bearing on the pin

$(DL+LL+W)=\frac{25000}{1+\frac{274.5}{406.8}}\times 1.3=19{,}400$ lbs. per sq. in. (From Eq. (21) and Spec. §39), and the allowed bearing pressure per linear inch of pin is $5\frac{3}{4}\times 19400=111{,}550$ lbs.

None of the bearing plates can however be counted on to take more stress in bearing than they transmit past the pin hole in tension.

The stress transmitted by the 2—14″×⅜″ plates past the pin hole will be proportional to the areas, and is $\frac{5.60}{29.25}\times 345{,}900=$ 66,200 lbs.

The stress transmitted by the other pin plates is 345,900—66,200=279,700 lbs.

Then the required bearing thickness of the 16 inch pin plates$=\frac{279700}{111550}=2.5$ inches, therefore the 2—16″×⅝″ plates on each side are sufficient.

The required net length back of the pin hole$=\frac{17.83}{3.25}=5.5$ in. Allowing one rivet hole out this will requrie the pin plates to extend $5\frac{1}{2}+1+2\frac{7}{8}=9\frac{3}{8}$ inches beyond the pin center.

The stresses taken by the component parts of the *body* of the member will be in proportion to their gross areas because their deformations must be equal, and the connection must distribute this stress properly to the component parts.

The stress taken in the body of the member by 1—14″×⅜″ plate$=\frac{5.25}{28.50}\times 345{,}900=63{,}700$ lbs.

The stress transmitted past the pin hole by each of the 16″×⅝″ pin plates$=\frac{1}{4}\times 279{,}700=69{,}980$ lbs., and sufficient rivets must be provided to transmit this stress from the pin plates to the body of the member.

The six countersunk rivets between the pin and the end of the angles may be considered as transmitting stress from the outside pin plate to the 14″×⅜″ plate so long as this does not raise the total stress in that plate beyond 63,700 lbs.

The value of the six countersunk rivets in the ⅜ inch plate is $6\times\frac{3}{7}\times 4{,}922=12{,}660$ lbs. (Spec., §40.) This would bring the total stress in the 14″×⅜″ plate at the end of the angles

up to $12{,}660+\frac{1}{2}(66200)=45{,}760$ lbs., which is less than 63,700 lbs., and therefore safe.

Further rivets will be required in the outside pin plate to transmit $69{,}980-12{,}660=57{,}320$ lbs. The rivets will be in single shear and the number required$=\frac{57320}{5412}=11$.

The rivets to transmit the stress from the inside $16''\times\frac{5}{8}''$ pin plate must be placed beyond those required for the outside pin plate. The number required$=\frac{69980}{5412}=13$.

The required net area of the body of the member through the first line of rivets of the connection plates is 22.29 sq. in., and the required net area on a zig zag line of holes$=1.3\times22.29=28.98$ sq. in. (Spec. §64.)

Assuming that a lacing rivet comes opposite the first rivet in the pin plate (which is not exactly the case here) we may write an equation as follows, and solve for the least allowable pitch of rivets in the connection plate at this point. (11). Calling this distance x we have

$$28.98=\tfrac{1}{2}\times4\left(1\tfrac{1}{2}+1\tfrac{1}{2}+\sqrt{(2\tfrac{1}{4})^2+x^2}+\sqrt{(3\tfrac{3}{4})^2+x^2}-3\right)$$
$$+2\times\tfrac{3}{8}\left(2\tfrac{1}{8}+2\tfrac{1}{2}+5\tfrac{1}{4}+2\sqrt{(2\tfrac{1}{4})^2+x^2}-4\right)$$

Solving we get $x=3\frac{1}{2}$ inches.

This pitch may safely be reduced to 3 inches after the first line of rivets is passed, as the stress in the body of the member has been reduced by the value of the rivets passed.

The lacing of a tension member does not have to comply with §97 of the specifications, and may be put in according to the judgment of the engineer.

76. Location of Pins in Top Chord and End Posts. (58). The location of the pins in the top chords and end posts depends upon the location of the centers of gravity of the sections and upon the amount of the displacement of the pins necessary to compensate for the bending due to the weight of the member.

The pin at the hip cannot be placed in the exact theoretical location for both the end post and top chord, and its location must necessarily be a compromise between the two.

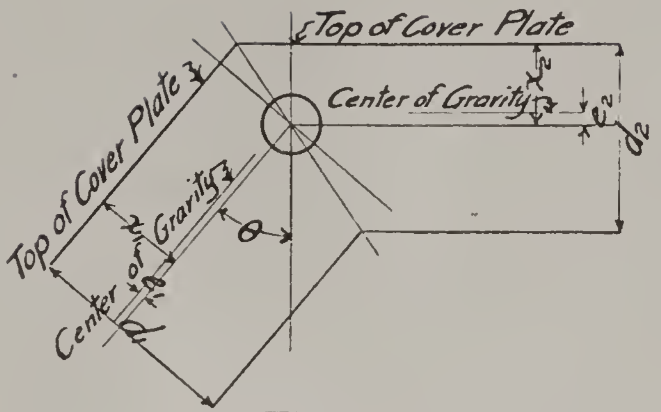


Fig. 73.

Figure 73 shows the factors which must be taken into consideration in this problem. From §43 of the specifications we see that the bending moment due to the weight of the member need not be considered unless it increases the fiber stress more than 10% above the allowed unit.

The weight of the end post will be as follows:

1	Cover plate	24″x⅝″	51.0
2	Web plates	21″x⅞″	89.2
2	Side plates	15″x⅝″	63.8
2	Top	*Ls* 3″x3″x⅝″	23.0
2	Bottom	*Ls* 4″x3″x⅝″	27.2
			254.2
	Details say 10%=		25.8
		Total=	280.0 lbs. per lin. ft.

The weight per linear foot horizontal will be 280 $sin\theta$=180 lbs., and the bending moment due to the weight will be

$$\frac{180\times27^2}{8}=16,400 \text{ ft. lbs.}$$

The maximum compressive fiber stress due to weight $=\frac{16400\times12\times9.48}{4698}=400$ lbs. per sq. in. in the top of the cover plate. The allowed unit stress for $DL+LL$ is 6516 lbs. per sq. in., therefore the bending moment due to weight need not be considered in the end post.

The maximum compressive fiber stress due to weight or to displacement of the pins may reach 651 lbs. per sq. in.

The weight of the end section of the top chord is as follows:

1	Cover plate	24″x½″	40.8
2	Web plates	18″x $\frac{9}{16}$″	68.8
2	Top	*Ls* 3″x3″x⅜″	14.4
2	Bottom	*Ls* 4″x3″x $\frac{\ }{16}$″	24.8
			148.8
	Details say 10%=		14.2
			163.0 lbs. per ft.

The bending moment due to the weight $=\frac{163\times27^2}{8}=14{,}880$ ft. lbs. The distance from the middle of the web to the center of gravity is 1.98 in., and the moment of inertia about the horizontal axis is 2230.

The maximum compressive fiber stress due to weight$=$ $\frac{14880\times12\times7.64}{2230}=612$ lbs. per sq. in.

The average allowed unit stress for $DL+LL$ from equation (21) is 9490 lbs. per sq. in., therefore the bending due to weight need not be considered in the top chord.

The most desirable location for the pin will be obtained from equation (20) as follows:

$$\text{For the end post } e=\frac{180\times27^2\times12}{10\times378500}=0.42 \text{ in.}$$

$$\text{For the top chord } e=\frac{163\times27^2\times12}{10\times406800}=0.36 \text{ in.}$$

For the end post the pin should be $1.77-0.42=1.35$ in. above the center line of the web.

For the top chord the pin should be $1.98-0.36=1.62$ in. above the center line of the web.

If x_2 in Fig. 73 is made 8 in. to agree with the most desirable position for the top chord, from similar triangles $x_1 : x_2 = d_1 : d_2$ or $x_1=9.33$ in., which would place the pin in the end post above the center of gravity.

If x_1 is made 9½ in. to agree with the most desirable position for the end post, $x_2=8.14$ in.

This will place the pin 1¾ in. above the center of the web of the end post and 1½ in. above the center of the web of the top chord. This location will be used.

The pins at the intermediate top chord points are placed on the same center lines at those at the hips, as they only have to transmit the increments of stress from the diagonals.

Figure 74 is a detail of the hip joint.

77. Lacing of Compression Members. (58). For the top chord CD (which is the largest) the allowed unit stress for $DL+LL$ from equation (21) is $\frac{20000-90\frac{L}{r}}{1.675}=11{,}940-53.7\frac{L}{r}=$ 9,920 lbs. per sq. in. (using the radius of gyration about the vertical axis.)

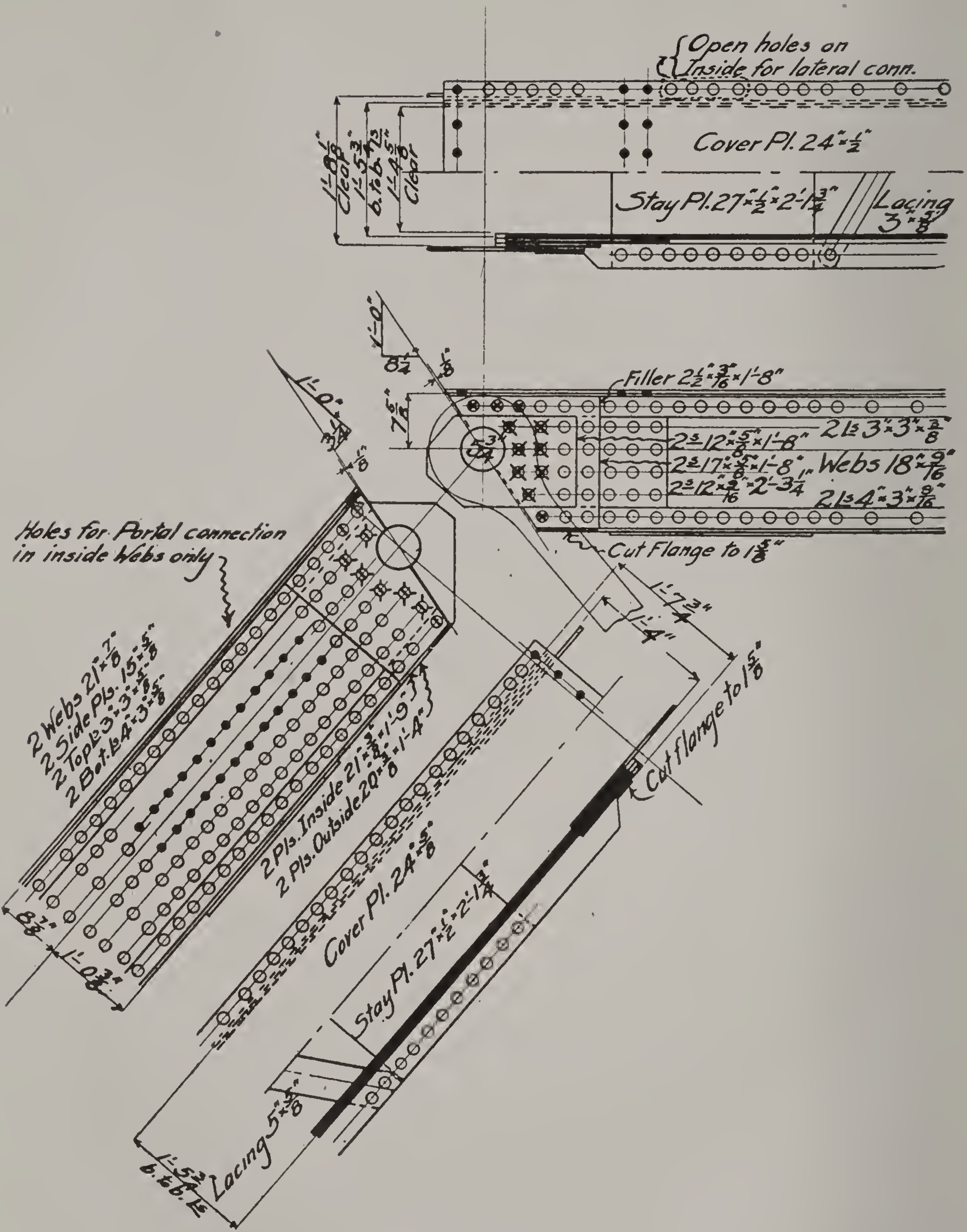


Fig. 74.

From equation (18).

$$f = \frac{4 \times 19.93 \times 20}{27 \times 12} = 497 \text{ lbs. per inch.}$$

Length of member covered by one lace bar (single lacing, Spec. §97)=22¼ *cot.* 60°=12.8 in.

Longitudinal increment of stress taken by one bar=497× 12.8×½=3180 lbs. (Half is taken by cover plate.)

Stress in one bar=3180 *sec.* 60°=6360 lbs.

In the lattice it is safe to use the unit stresses allowed for lateral bracing.

Specifications §97 requires the lattice for this chord to be ⅝ in. thick.

Allowed unit stress in compression is

$$13000-60\ \frac{L}{r}=13000-\frac{60\times 25.6}{0.181}=4520 \text{ lbs. per sq. in.}$$

$$\text{Required area}=\frac{6360}{4520}=1.41 \text{ sq. in.}$$

$$\text{Required width}=\frac{1.41}{.625}=2.26 \text{ in.}$$

Use lace bars 3″x⅝″ for top chords to comply with specifications §97.

The lacing for the end posts cannot be obtained directly from equation (18) because the end post carries transverse shear in addition to the direct stress.

The total difference in extreme fiber stress due to column action is obtained from the column formula for $DL+LL+$ Wind. From Eq. (21)

$$s_c=\frac{17000-90\dfrac{L}{r}}{1.675}\times 1.3=13{,}200-69.8\ \frac{L}{r}$$

The values of L and r here must be taken about a vertical axis because we are figuring for the shear in that direction.

The difference in unit stresses due to column action$=69.8\times$ $\dfrac{34\times 12}{8.68}=3280$ lbs. per sq. in.

The difference in unit stresses due to transverse bending from Art. 66 = 3310 lbs. per sq. in.

Total difference=3310+3280=6590 lbs. per sq. in.

Total stress to be transferred by the lacing and cover plate in a distance of 17 ft. (distance from end to point of contraflexure)$=6590\times A_1=6590\times 35.1=231{,}300$ lbs.

$$f=\frac{231300}{17\times 12}=1134 \text{ lbs. per inch.}$$

Longitudinal increment of stress taken by one bar$=1134\times 12.8\times \frac{1}{2}=7260$ lbs.

Total stress in one bar=7260 *sec.* 60°=14,520 lbs.

Using bars $\frac{5}{8}$ in. thick, the allowed compressive unit stress is 4520 lbs. per sq. in.

$$\text{Required area}=\frac{14520}{4520}=3.21 \text{ sq. in.}$$

$$\text{Required width}=\frac{3.21}{.625}=5.12 \text{ in.}$$

Use lace bars $5'' \text{x} \frac{5}{8}''$ with two rivets in each end.

For the intermediate posts Cc, the radius of gyration perpendicular to the channel webs is 4.31 in, and the unsupported length in that direction about 21 ft.

The allowed unit stress for $DL+LL=10{,}060-53.25\,\frac{L}{r}$

(See Art. 65) $=6950$ lbs. per sq. in.

From equation (18) we get

$$f=\frac{4\times 11.76\times 3110}{21\times 12}=580 \text{ lbs. per inch.}$$

The length covered by one lace bar (single lacing, Spec. §97) is $3\frac{1}{2}$ inches.

Longitudinal increment of stress taken by one bar$=580\times 3\frac{1}{2}\times\frac{1}{2}=1015$ lbs. (Lacing on two sides.)

Stress in one bar$=1015$ *sec.* $60°=2030$ lbs.

Specifications §97 requires that the lacing for this case be $\frac{7}{40}$ inches thick, but §82 limits us to $\frac{3}{8}$ in.

$$\text{Allowed unit stress}=13000-\frac{60\times 7}{.108}=9100 \text{ lbs. per sq. in.}$$

$$\text{Required area}=\frac{2030}{9100}=0.23 \text{ sq. in.}$$

$$\text{Required width}=\frac{.23}{.375}=0.61 \text{ in.}$$

Specifications §97 requires $2\frac{1}{2}$ in. $\times$ $\frac{3}{8}$ in.

For post Dd specifications requires lacing $2\frac{1}{4}'' \text{x} \frac{3}{8}''$.

78. Details of the Floor Beams. Figure 75 shows a detail drawing of one of the intermediate floor beams. Cooper's specifications requires the use of a number of different unit stresses for rivets in various positions, and these must be kept in mind. (§40)

$$\text{Rivets required for stringer connection}=\frac{90800}{2625}=35.$$

Rivets required for end connection angles through web $=\frac{90800}{8659}=11$.

Rivets required for end connection to post$=\frac{90800}{2886}=32$.

Rivets required for connection of flanges to web$=\frac{86100}{3938}=23$.

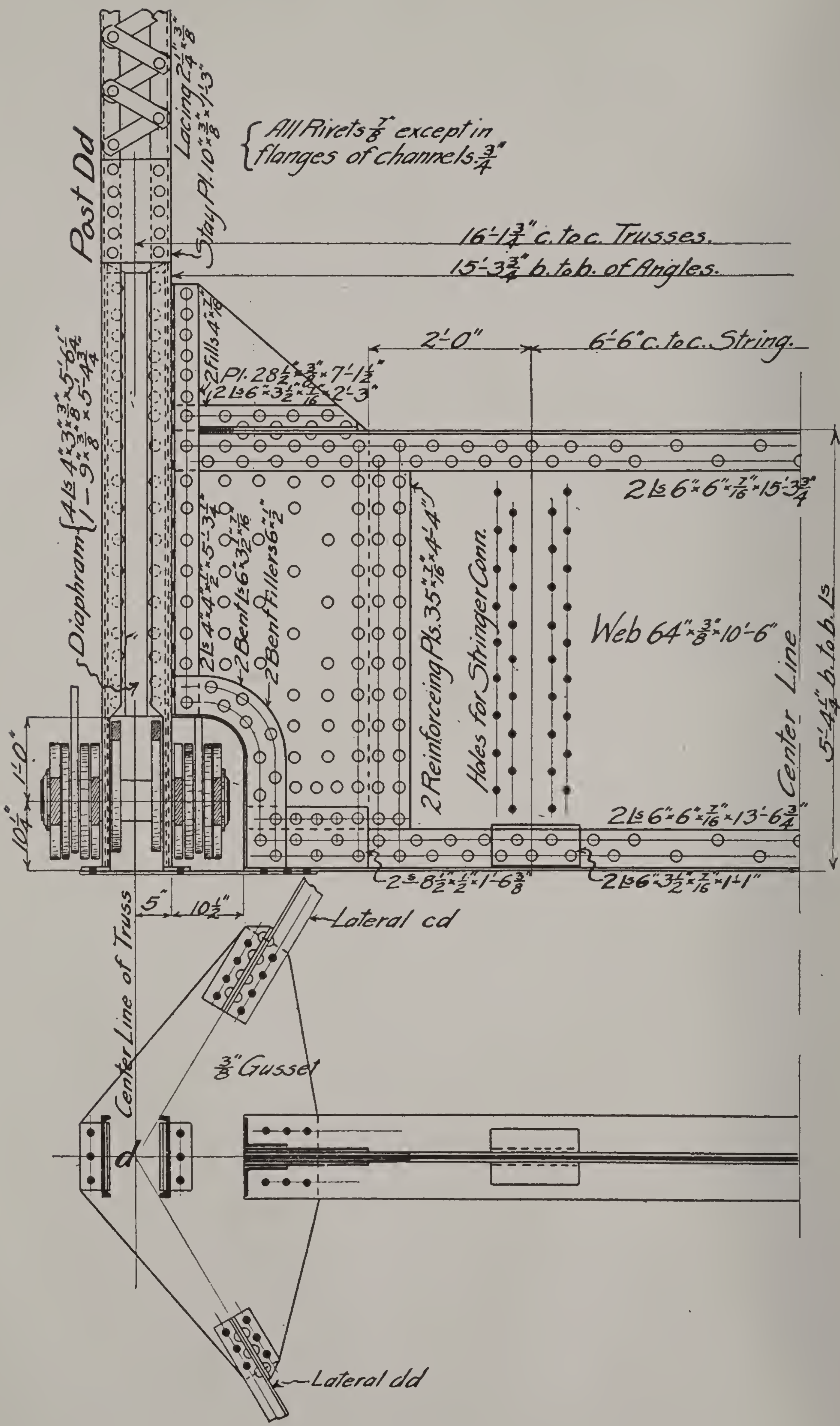
(Shop rivets in bearing on ⅜ in. web.)

At the end of the bottom flange the expedient is resorted to of riveting a plate on top of the angles to transfer a part of the stress to the web. This then gives us the following value:

16 rivets bearing on ⅜ in. web	$=16\times3938=$	63,000 lbs.
6 rivets double shear	$=6\times8659=$	51,900 lbs.
		114,900 lbs.

The top flange has 26 rivets effective.

Rivets required for web splice$=\frac{90800}{3938}=23$.



Post Dd
Lacing $2\frac{1}{4}" \times \frac{3}{8}"$
Stay Pl. $10" \times \frac{3}{8}" \times 1'$-$3"$
All Rivets $\frac{7}{8}"$ except in flanges of channels $\frac{3}{4}"$
16'-$1\frac{3}{4}"$ c. to c. Trusses.
15'-$3\frac{3}{4}"$ b. to b. of Angles.
2'-0"
6'-6" c. to c. String.
2 Fills $4" \times \frac{7}{16}"$
Pl. $28\frac{1}{2}" \times \frac{3}{8}" \times 7'$-$1\frac{1}{2}"$
2 Ls $6" \times 3\frac{1}{2}" \times \frac{7}{16}" \times 2'$-$3"$
Diaphram
4 Ls $4" \times 3" \times \frac{3}{8}" \times 5'$-$6\frac{1}{4}"$
1-$9" \times \frac{3}{8}" \times 5'$-$4\frac{3}{4}"$
2 Ls $4" \times 4" \times \frac{1}{2}" \times 5'$-$3\frac{1}{4}"$
2 Bent Ls $6" \times 3\frac{1}{2}" \times \frac{7}{16}"$
2 Bent Fillers $6" \times \frac{1}{2}"$
2 Reinforceing Pls. $35" \times \frac{7}{16}" \times 4'$-$4"$
Holes for Stringer Conn.
2 Ls $6" \times 6" \times \frac{7}{16}" \times 15'$-$3\frac{3}{4}"$
Web $64" \times \frac{3}{8}" \times 10'$-$6"$
Center Line
5'-$4\frac{1}{4}"$ b. to b. Ls
2 Ls $6" \times 6" \times \frac{7}{16}" \times 13'$-$6\frac{3}{4}"$
1'-0"
$10\frac{1}{4}"$
2-$8\frac{1}{2}" \times \frac{1}{2}" \times 1'$-$6\frac{3}{8}"$
2 Ls $6" \times 3\frac{1}{2}" \times \frac{7}{16}" \times 1'$-$1"$
5"
$10\frac{1}{2}"$
Lateral cd
Center Line of Truss
$\frac{3}{8}"$ Gusset
d
Lateral dd

Fig. 75.

INDEX.

	Art.	Page
Adding machines	18	37
Angles—maximum lengths	52	124
Bearing plates for plate girder	52	132
Bending of chords due to weight	58	145
Bolsters for plate girder	52	132
Books of reference	25	50
Bottom lateral bracing	70	170
Box girders	42	99
Building construction	37	80
Built tension members	57	141
tension members	64	152
Butt joints	8	12
Button head rivets	1	1
Calculation of pins	74	178
Center of gravity	65	157
Chords—Design of	65	156
—Location of pins	76	185
Classes of structural steel work	14	29
Clearances	27	60
Columns	58	142
Compression and bending combined	58	144
Compression members	58	142
—Allowance for rivet holes	4	5
—Design of	65	154
—Width of	65	155
Contracts and proposals	16	30
Corrugated iron or steel	35	77
Countersunk rivets	1	1
Countersunk rivet values	9	15
Dead load	61	147
for plate girder bridge	52	116
of truss bridges	56	141
Dead loads—Roof	40	86
Deck plate girder bridge design	52	115
estimate	52	134
stress sheet	52	136
Depth of truss	62	147
Depth of trusses	55	139

	Art.	Page
Design of chords	65	156
compression members	65	154
end posts	66	159
deck plate girder bridge	52	115
floor beam	68	165
pin-connected bridge	60	146
portal	67	163
riveted connections	9	13
a roof	40	84
a stringer	51	110
tension members	64	150
Designing and estimating	17	31
Designing and estimating	Chap. II	29
Details—Duplication of	27	58
floor beams	78	190
hip joint	76	188
pin connected bridges	Chap. VII	177
a riveted tension member	75	182
Dolly	3	4
Drafting department	24	46
Draftsman's equipment	25	47
Drawing boards	24	47
instruments	25	48
pencils	25	49
room light	24	47
table	24	47
Drawings for roof trusses	41	91
Drawing of roof truss	..	98
Drift pins	2	3
Driving rivets	3	3
Eccentricity of pins in chord members	58	145
Economic depth of girders	46	103
girders	51	111
girders	52	119
trusses	55	139
Effective depth of plate girders	45	103
End floor beam	68	167
End posts—Design of	66	159
—Lacing	77	189
Equivalent loads	56	140
Erection	23	45
Erection and manufacture	Chap. III	44
Estimate for deck plate girder bridge	52	134
for pin connected bridge	72	173
of weight of stringers	51	115
Estimates of cost	17	31
Estimates—Forms of	17	33

	Art.	Page
Estimating and designing	17	31
and designing	Chap. II	29
—Order of	19	37
—Order for highway bridges	19	39
—Order for railway bridges	19	38
—Order for steel buildings	19	40
Eye bars	57	141
False work	23	45
Field riveting	23	46
Field rivet values	9	14
Flange plates of plate girders	52	121
riveting in plate girders	49	107
riveting in plate girders	51	113
riveting in plate girders	52	122
sections	42	99
splices in plate girders	50	110
splices in plate girders	52	124
Flanges of girders	45	102
girders	51	112
girders	52	120
Floor beams—Details	78	190
—Details	78	192
—Design	68	165
Floor—Design of, for bridge	52	115
Freight	31	69
Gallows frame	23	45
General plans	21	42
Girders	Chap. V	99
—Economic depth	46	103
—Effective depth	45	103
—The flanges	45	102
—Flange riveting	48	107
—Flange splices	50	110
—Maximum lengths	42	99
—Moment of resistance	43	99
—Shear distribution	44	100
—Stiffeners	47	105
—Stresses	43	99
—Stresses	51	112
—Stresses	52	117
—The web	44	100
—Web splices	48	106
Grip of rivets	1	1
Heating rivets	3	4
Hip joint—Detail of	76	188
pin	74	178
Inertia—Moment of	65	158

	Art.	Page
Inspection	33	75
Intermediate posts—Lacing	77	190
Jack rafters	37	80
Joints—Butt	8	12
—Lap	8	12
Lacing of compression members	58	143
compression members	77	187
Lap joints	8	12
Laterals—Bottom	70	170
Lateral bracing of plate girders	52	127
stringers	51	114
Lateral systems	59	146
Laterals—Top	69	169
Loads	56	140
Loads—Roof	38	81
Location of pins in top chord and end posts	76	185
Manufacture and erection	Chap. III	44
Materials	32	70
Material orders	26	51
Moment in deck plate girder	52	117
Moment of inertia	65	158
Moment of resistance of plate girder	43	100
Net sections of tension members	11	20
Order of estimating	19	37
highway bridges	19	39
railway bridges	19	38
steel buildings	19	40
Order of procedure for a pin connected bridge	28	62
for plate girder bridge	29	65
Ordering material	26	51
Packing	73	178
Packing at various joints	74	182
Panel lengths	55	139
Pins	73	177
Pins—Calculation	74	178
Pin-connected bridges	Chap. VI	139
—Design of	60	146
details	Chap. VII	177
—Estimate	72	173
—Order of procedure for	28	62
stresses	63	148
—Stress sheet	72	176
Pins—Location in top chord and end posts	76	185
Pin plates	10	16
Pin plates	10	18
Pitch of rivets defined	7	10
Pitch of roofs	35	78

	Art.	Page
Plans—General	21	42
Plans—Show	21	43
Plate girder bridges	Chap. V	99
Plate girder bridge—Order of procedure	29	65
Plate girder design	52	115
Plate girders—Economic depth	46	103
—Economic depth	51	111
—Economic depth	52	119
—Effective depth	45	103
—Effective depth	51	112
—Effective depth	52	120
—Flanges	45	102
—Flanges	51	112
—Flanges	52	120
—Flange plates	52	121
—Flange riveting	49	107
—Flange riveting	51	113
—Flange riveting	52	122
—Flange splices	50	110
—Flange splices	52	124
—Lateral bracing	52	127
—Maximum moment	52	117
—Stiffeners	47	105
—Stiffeners	51	114
—Stiffeners	52	124
—Stresses	43	99
—Stresses	51	112
—Stresses	52	117
—Web	44	100
—Web	51	111
—Web	52	120
—Web splices	48	106
—Web splices	52	120
Plates—Sizes of	26	53
Pony trusses	55	140
Portal bracing	67	163
Proposals and contracts	16	30
Purlins	40	86
Radius of gyration	65	157
Rafter design, for a roof truss	40	88
Reaming	2	2
Rivet holes	2	2
—Allowance for in compression members	4	5
in tension members	11	20
Rivet pitch in flanges of girders	49	107
Rivet spacing	27	59
Riveted joints—Alternating stresses	4	7

	Art.	Page
Riveted joints—Alternating stresses	5	9
—Assumption made in design of......	4	5
—Design of	9	13
—Examples	10	15
—Friction in	4	6
—Kinds of	8	12
—Manner of failure	9	13
—Requirements for good	5	9
—Slip in	4	6
Riveted tension member details	75	182
Riveted truss bridges	28	64
Riveting	Chap. I	1
—Chain	9	13
—Design of joints in a roof truss..........	41	93
of flanges of plate girders................	49	107
of flanges of plate girders................	51	113
of flanges of plate girders................	52	122
machines	3	3
of pin plates	10	16
of pin plates	10	18
—Staggered	9	13
—Theory of	4	4
Rivets—American Bridge Company's standard......	1	1
—Bending in	4	7
—Button heads	1	1
—Conventional signs for	13	28
—Countersunk	1	1
—Dimensions of	1	1
—Driving	3	3
—Field values	9	14
—Grip	1	1
—Heating	3	4
—Initial tension in	4	6
—Length required	1	2
—Shape of heads	1	1
—Proper sizes	6	9
—Spacing of	7	10
—Values of countersunk	9	15
—Working stresses	9	14
Rollers	71	171
Roofs	Chap. IV	77
construction	34	77
coverings	35	77
—Dead load	40	86
—Design of a	40	84
loads	38	81
pitch for various coverings	35	78

	Art.	Page
Roof purlins	40	86
truss drawings	41	91
truss drawing	41	98
trusses—Types of	36	78
Scales	25	49
for roof truss drawing	41	92
for shop drawing	28	63
Shear—Distribution over cross section of girder	44	100
Shear in girders	44	100
Shear in girders	51	111
Shear in girders	52	118
Shipment	31	69
Shoes for plate girder	52	132
Shoes and rollers	71	171
Shop bills	30	67
Shop drawings	27	54
drawings—Methods of working up	27	61
drawings—Notes on	27	58
drawings for roof truss	41	91
drawings for roof truss	41	98
drawings—Titles on	27	56
Shop operations	22	44
Shops—Kinds of	15	30
Show plans	21	43
Signs for rivets on drawings	13	28
Sizes of rivets	1	1
Sizes of rivets	6	9
Slide rules	18	35
—Duplex	18	36
—Engineer's	18	36
—Fuller's	18	35
—Manheim	18	35
—Rule for operation of	18	36
—Thacher's	16	35
—Three multiple	18	36
Snap for rivet heads	3	4
Snow loads	38	83
Solid floors	53	138
Spacing of rivets	7	10
Specifications	20	41
Splices in flanges of plate girders	52	124
—Flanges of girders	50	110
in webs of plate girders	48	106
in webs of plate girders	52	125
Steel—Acid	32	72
—Basic	32	72
—Bessemer	32	71

	Art.	Page
Steel—Effect of carbon	32	71
—Open hearth	32	72
—Physical characteristics	32	70
—Process of manufacture	32	71
—Specifications for	32	70
—Tests of	32	74
Stiffeners	29	65
Stiffeners	47	105
Stiffeners	51	114
Stiffeners	52	124
Stiffeners—Crimped	47	105
Stress sheets	21	42
for deck plate girder bridge	52	136
for pin connected bridge	72	176
for a roof	40	90
Stresses in girders	43	99
in girders	51	112
in girders	52	117
in pin connected bridge	63	148
in roof trusses	39	84
in roof trusses	40	87
in trusses of bridge due to wind	63	149
Stringer—Design	51	110
—Estimate of weight	51	115
laterals	51	114
Structural steel—Classes of	14	29
Templets	22	44
Tension members	57	141
—Built	64	152
—Details of riveted	75	182
—Design of	64	150
—Net sections	11	20
Tension on rivet heads	4	6
Tests of steel	32	74
Theory of riveting	4	4
Through plate girders	53	137
Ties—Design of for bridge	52	115
Time savers	18	35
Titles on shop drawings	27	56
Top chord—Lacing	77	187
Top lateral bracing	69	169
Tracing linen	27	54
Traveler	23	45
Trusses—Types of	55	139
Types of plate girder flanges	42	99
Types of roof trusses	36	78
Types of trusses	55	139

	Art.	Page
Webs of Girders	44	100
of girders	51	111
of girders	52	120
—Moment of resistance	44	101
—Moment of resistance	45	102
splices in plate girders	48	106
splices in plate girders	52	125
Width of compression members	65	155
Wind pressure	38	82

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